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Existing Resources, Standards, and Procedures for Precise Monitoring and Analysis of Structural Deformations - Volume I

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13. ABSTRACT (Maximum 200 words)

This report is based on a review of literature, international reports, and results of a questionnaire sent to national representatives of about 70 countries to the International Commission on Large Dams. The study focused on monitoring and analysis of deformations of large dams. The main conclusions of the study are: (1) There are no available standards and specifications in any of the reviewed countries which could be recommended for direct adaptation to dam deformation monitoring in the United States; (2) With the recent technological developments in both geodetic and geotechnical instrumentation, at a cost one may achieve almost any, practically needed, instrumental resolution and precision, full automation, and virtually real-time data processing; (3) Over the past 10 years there has been significant progress in the development of new methods for the geometrical and physical analyses of deformation surveys. However, due to a lack of an interdisciplinary cooperation and insufficient exchange of information, the developments have not yet been widely adapted in practice; and (4) Generally, the overall qualifications and educational background of the personnel placed in charge of monitoring surveys in the U.S. seem to be inadequate, particularly in the areas of data processing and analyses.

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PREFACE

This report was prepared under Contract DAAL03-91-C-0034 for the U.S. Army Topographic Engineering Center, Fort Belvoir, Virginia 22050-5546 under the auspices of the U.S. Army Research Office, Scientific Services Program, Research Triangle Park, NC 27709, by the Department of Surveying Engineering, University of New Brunswick, Frederiction, N.B., E3B 5A3, Canada. The Contracting Officer's Representative was Ms. Sally L. Frodge.

SUMMARY

The following major tasks have been assigned by TEC for the preparation of this report:
(1) Describe existing standards and procedures employed throughout the world for precise deformation surveying and monitoring work;
(2) Describe existing software and automated products that exist for precise deformation surveying and monitoring work that are commercially available or under prototype development;
(3) Describe instrumentation and observation methods that are used and employed in this work area.

To fulfill the requirements of the assigned tasks, the study (focussed on deformation of large dams), has been performed based mainly on available recent literature concerning the monitoring and analysis of deformations; reports by the International Commission on Large Dams (ICOLD) and the International Federation of Surveyors; and personal experience of the authors. In addition, a questionnaire was sent to over 70 national Committees on Large Dams (members of ICOLD) requesting information on the existing standards and specifications for dam monitoring, status of dam monitoring in their countries, and methods used.

The major conclusions of the study are:
(1) There is no one country which can serve as an example for others concerning all three main aspects of dam deformation monitoring, i.e. monitoring techniques, design of monitoring schemes, and analysis and management of the collected observations.
(2) There are no available standards and specifications in any of the reviewed countries which could be recommended for direct adaptation to dam deformation monitoring in the United States.
(3) With the recent technological developments in both geodetic and geotechnical instrumentation, at a cost one may achieve almost any, practically needed, instrumental resolution and precision, full automation, and virtually real-time data processing.
(4) Over the past 10 years there has been a significant progress in the development of new methods for the geometrical and physical analyses of deformation surveys. However, due to thelack of an interdisciplinary cooperation and insufficient exchange of information, the developments have not yet been widely adopted in practice.
(5) Generally, the overall qualifications and educational background of the personnel placed in charge of monitoring surveys in the US seem to be inadequate, particularly in the areas of data processing and analyses.

Several recommendations are given to improve the situation.

1. INTRODUCTION

1.1 BACKGROUND INFORMATION

Deformation monitoring, analysis, and prediction are of a major and ever growing concern in practically all fields of engineering and geoscience. Safety, economical design of man-made structures, efficient functioning and fitting of structural elements, environmental protection, and development of mitigative measures in the case of natural disasters (land slides, earthquakes, liquefaction of earth dams, etc.) require a good understanding of causes (loads) and the mechanism of deformation which can be achieved only through the proper monitoring and analysis of deformable bodies.

The development of new methods and techniques for the monitoring and analysis of deformations and the development of methods for the optimal modelling and prediction of deformations have been the subject of intensive studies by many professional groups at national and international levels. Within the most active international organizations which are involved in deformation studies one should list:

- International Federation of Surveyors (FIG) with its Study Group 6C which has significantly contributed to the recent development of new methods for the design and geometrical analysis of integrated deformation surveys and new concepts for global integrated analyses and modelling of deformations;
- International Commission on Large Dams (ICOLD) with its Committee on Monitoring of Dams and their Foundations;
- International Association of Geodesy (IAG) with the very active Commission on Recent Crustal Movements which frequently organizes international and regional symposia concerning geodynamics, tectonic plate movements, and modelling of regional earth crust deformation.
- International Society for Mine Surveying (ISM) with their very active Commission 4 on Ground Subsidence and Surface Protection in mining areas;

- International Society for Rock Mechanics (ISRM) with their overall interest in rock stability and ground control; and
- International Association of Hydrological Sciences (IAHS) which organizes international symposia (e.g., Venice 1984, Houston 1991) on ground subsidence due to the withdrawal of underground liquids (water, oil, etc.).

The FIG Study Group 6C has been one of the most, if not the most, active international groups dealing with practically all aspects of deformation monitoring and analysis. Since 1975, the FIG Study Group 6C has organized six international symposia with the last symposium held in Hannover in 1992 and the 7th symposium already being planned for Banff, Canada, in May 1993. Although the activity of FIG in the development of new monitoring techniques is biased, of course, towards geodetic surveying techniques, its activity in the design and analysis of deformation surveys is more objective than that of any other professional group. In 1978, an *ad hoc* Committee on Deformation Analysis was formed to deal with and clarify various approaches and schools of thinking regarding the geometrical analysis of deformation surveys including the identification of unstable reference points. The work of the Committee has resulted in the development of new concepts for integrated monitoring systems (integration of geodetic and geotechnical measurements) and for generalized global analyses of integrated deformation surveys. The work of the *ad hoc* committee has been summarized in four progress reports (Chrzanowski et al., 1981; Heck et al., 1982; Chrzanowski and Secord, 1983; Chrzanowski and Chen, 1986) and the final report (Chrzanowski and Chen, 1990), presented at the XIXth FIG Congress in Helsinki in 1990 (Appendix 1).

Most of the activities and studies of other associations and organizations focus, of course, on direct applications to their particular deformation problems. Although the accuracy and sensitivity criteria for the determination of deformation may considerably differ between various applications, the basic principles of the design of monitoring schemes and their geometrical analysis remain the same. For example, a study on the stability of magnets in a nuclear accelerator may require determination of relative displacements with an accuracy of 0.05 mm while a settlement study of a rock-fill dam may require only 10 mm. In both cases, although the

monitoring techniques and instrumentation may differ, one may show that the same basic methodology in the designing and analysis of the deformation measurements may be utilized. This fact has not yet been fully understood by most of the above listed international research groups. There is a general lack of communication and work coordination. The studies of various professional groups, not only at the international but also at the national level of individual countries, overlap resulting in the duplication of efforts in 'discovering' methods and techniques which have already been well known to other study groups. For example, in the United States there is very little communication between the U.S. Bureau of Reclamation, U.S. Army Corps of Engineers, U.S. Commission on Large Dams, and the American Congress on Surveying and Mapping. All four organizations are involved separately in studies on deformations of engineering structures and in the development of guidelines for deformation monitoring. Even within the same organization or institution, one may find examples of two different professional groups, for instance geotechnical and survey engineers, who may work on the same deformable object but do not exchange information on their methods and the results of their analyses.

The above problems have been recognized by the Topographic Engineering Center (TEC) of the U.S. Army Corps of Engineers, who came to the conclusion that a multi-disciplinary effort is needed in elaborating unified guidelines for the monitoring and analysis of structural deformations. As an initial step, a research group led by Dr. A. Chrzanowski at the University of New Brunswick (UNB) in Canada was approached by TEC, represented by Sally L. Frodge, in April, 1992, to prepare a preliminary report on the existing resources, standards, and procedures for structural deformation surveys. Over the last 20 years, the UNB Group has actively participated in the studies of various international organizations (FIG, IAG, ISM, and ISRM) involved in deformation surveys. Since 1978, Dr. A. Chrzanowski has been chairman of the aforementioned FIG *ad hoc* Committee on Deformation Analysis and, since 1988, is chairman of the FIG Study Group 6C on Deformation Measurements. The FIG Study Group, not being specialized in any particular engineering or geoscience application of the results of the deformation studies, deals objectively with all general aspects of optimal monitoring and analysis which are applicable to any deformation problem.

1.2 SCOPE OF WORK

The following specific tasks have been assigned by TEC for the preparation of this report:

(1) Describe existing standards and procedures employed throughout the world for precise deformation surveying and monitoring work;

(2) Describe existing software and automated products that exist for precise deformation surveying and monitoring work that are commercially available or under prototype development;

(3) Describe instrumentation and observation methods that are used and employed in this work area.

Structural deformation problems cover a very wide spectrum of engineering, from deformations of underground excavations and tunnels, through deformations of dams, dykes, and other sensitive man-made structures, to land slides and ground subsidence problems. In order to review thoroughly the status of monitoring techniques and observation methods used in all those applications one would have to employ a very labour intensive and prolonged study including information from all the above listed associations and member countries. However, due to financial and timing restrictions imposed by the Topographic Engineering Centre for the preparation of this report, the study had to be completed over a period of four months utilizing only 40 person-days. Therefore, the study for this report had to be either very general and superficial or narrowed to only one type of application. A compromised solution has been adopted by keeping the description of the methods of the deformation analysis and some comments relating to monitoring techniques as general as possible but narrowing the examples of practical applications as well as the description of the worldwide status of structural deformation to large dams only. Large dams (higher than 15 metres, as classified by ICOLD) belong to the most sensitive engineering structures. Thus the methods and techniques used in dam monitoring and analysis represent very well the methods and techniques used on any large engineering structure.

According to the recent information (dated September 3, 1992) obtained from the U.S. Committee

on Large Dams, the U.S. Army Corps of Engineers is in charge of 475 large dams out of a total of 5469 large dams maintained currently in the United States. An additional 49 large dams are currently under construction.

1.3 APPROACH TO THE STUDY AND ORGANIZATION OF THE REPORT

The study has been based mainly on the available recent literature on the monitoring and analysis of deformations (mainly ICOLD Bulletins and reports of the FIG Study Group on Deformation Measurements), and personal experience of the authors. In addition, a questionnaire was sent to over 70 national Committees on Large Dams (members of ICOLD) requesting the following information:

(a) Updated number of existing and being constructed large dams. How many of them are being monitored using geodetic and/or geotechnical/structural instrumentation?

(b) Existing national and/or local standards and specifications for dam monitoring.

(c) Publications either in technical journals or in any conference proceedings which describe monitoring of dams in the given country.

About 30 countries, including all the major owners of dams, have responded to the questionnaire. The collected material has been summarized in this report in two main parts (chapters):

- Chapter 2: Methods and techniques currently available or under development for the monitoring and analysis of deformations, and
- Chapter 3: World wide status of monitoring and analysis of dam deformations

The two chapters are followed by concluding remarks and recommendations for the development of interdisciplinary guidelines for the monitoring and analysis of deformations. Some selected publications and reports have been appended to this report as a separate volume.

2. METHODS AND TECHNIQUES CURRENTLY AVAILABLE OR UNDER DEVELOPMENT FOR MONITORING AND ANALYSIS OF DEFORMATIONS

2.1 REVIEW OF MONITORING TECHNIQUES AND INSTRUMENTATION

2.1.1 Classification and General Review

The monitoring schemes include:

- monitoring of deformations, i.e., determination of the geometrical change in shape and dimensions and rigid body translations and rotations (absolute and/or relative) of the monitored object; and
- monitoring of acting forces (loads) and internal stresses which can either be measured directly or derived from measurements of temperature, pore water pressure, water level, seepage, etc.)

In addition, laboratory and/or in-situ determination of physical properties of the deformable material (e.g., moduli of elasticity, tensile strength, creep parameters, porosity, etc.) are necessary for a proper design and evaluation of the behaviour of the monitored structure. In seismically active areas, the monitoring schemes must include special instrumentation for measuring vibrations. Chapter 3 gives examples from various countries on the use of various types of instruments in dam monitoring.

This chapter reviews only the techniques used in monitoring the deformations although all the other above mentioned components of the monitoring schemes play an equally important role in the analysis and interpretation of deformations as discussed in 2.3 below.

The measuring techniques and instrumentation for geometrical monitoring of deformations have traditionally been categorized into two groups according to the two main groups of professionals

who use the techniques:

(1) geodetic surveys which include conventional terrestrial, photogrammetric, satellite positioning, and some special techniques (interferometry, hydrostatic levelling, alignment, and other), and

(2) geotechnical/structural measurements of local deformations using tiltmeters, strainmeters, extensometers, joint-meters, plumb-lines, etc.

Each type of measurement has its own advantages and drawbacks as discussed in Chrzanowski (1986). Geodetic surveys, through a network of points interconnected by angle and/or distance measurements, usually supply a sufficient redundancy of observations for the statistical evaluation of their quality and for a detection of errors. They give global information on the behaviour of the deformable object while the geotechnical measurements give very localized and, very frequently, locally disturbed information without any check unless compared with some other independent measurements. On the other hand, geotechnical instruments are easier to adapt for automatic and continuous monitoring than conventional geodetic instruments. Conventional terrestrial surveys are labour intensive and require skilful observers, while geotechnical instruments, once installed, require only infrequent checks on their performance. Geodetic surveys have traditionally been used mainly for determining the absolute displacements of selected points on the surface of the object with respect to some reference points that are assumed to be stable. Geotechnical measurements have traditionally been used mainly for relative deformation measurements within the deformable object and its surroundings. However, with the technological progress of the last few years, the differences between the two techniques and their main applications are not as obvious as twenty years ago. For example, inverted plumb-lines and borehole extensometers, if anchored deeply enough in bedrock below the deformation zone, may serve the same way as, or even better than, geodetic surveys for determining the absolute displacements of the object points. Geodetic surveys with optical and electro-magnetic instruments (including satellite techniques) are always contaminated by atmospheric (tropospheric and ionospheric) refraction which limits their positioning accuracy to about ±1 ppm to ±2 ppm (at the standard deviation level) of the distance. So, for instance, with the average distance between the object and reference points of 500 m, the absolute displacements of the object points

cannot be determined with an accuracy better than about ±2 mm at the 95% probability level. In many cases this accuracy is not adequate. On the other hand, precision electro-optical geodetic instruments for electronic distance measurements (EDM) with their accuracies of ±0.3 mm over short distances, may serve as extensometers in relative deformation surveys. Similarly, geodetic levelling, with an achievable accuracy of better than ±0.1 mm over distances of 20 m may provide better accuracy for the tilt determination (equivalent to ±1 second of arc) than any local measurements with electronic tiltmeters. New developments in three-dimensional coordinating systems with electronic theodolites may provide relative positioning in almost real time to an accuracy of ±0.05 mm over distances of several metres. The same applies to new developments in photogrammetric measurements with the solid state cameras (CCD sensors). The satellite Global Positioning System (GPS), which, if properly handled, offers a few millimetres accuracy in differential positioning over several kilometres, is replacing conventional terrestrial su[illegible]s in many deformation studies (Chrzanowski et al., 1990c) and, particularly, in establishin[illegible] reference networks.

There are several, reasonably up-to-date, books on instrumentation. Among the best which is dealing with all types of geotechnical measurements (deformation and acting forces) is the book by Dunnicliff (1988) in which excellent remarks and practical guidelines on the optimal installation and use of various instruments are given. Another, though not so well organized, is a book by Hannah (1985) which gives over 800 references on geotechnical measurements. As far as main references on geodetic instrumentation are concerned, one should list the book by Kahmen and Faig (1988), and books on electronic distance measurements (Rueger, 1990; Burnside, 1991). ICOLD Bulletins N°. 41 (ICOLD 1982), no. 60 (ICOLD, 1988) and no. 68 (ICOLD, 1989) give good guidelines on the techniques and instrumentation for monitoring large dams. The U.S. Bureau of Reclamation (USBR) has produced instrumentation manuals for monitoring concrete dams (Bartholomew et al., 1987) and for embankment dams (Barthomolew and Haverland, 1987). Both publications give ample instrumentation information as used by USBR; however, some expert geotechnical engineers have reservations using these manuals as general guidelines for monitoring. Also, the USBR publications do not provide much guidance for the use of geodetic techniques.

A full review of all the instruments and techniques available for deformation monitoring would far exceed the scope of this report. In view of the available references listed above, the discussion below is limited only to some comments on the achievable accuracy of the basic types of instruments which include electronic instrumentation for distance and angle measurements, surveying robots, measurements of tilt and inclination, changes in distances and strain, photogrammetric methods, and the satellite Global Positioning System. In addition, more details on terrestrial geodetic and geotechnical basic measuring techniques are given in Appendices 2 and 3 respectively. It has been assumed that the readers of this report possess the basic understanding of geodetic and geotechnical measuring techniques and do not require explanations of what is a theodolite, or a hydrostatic level, or a tiltmeter. Some explanations are given in Appendices 1 and 2. Additional information can be found in the above references.

2.1.2 Electronic Distance and Angle Measurements

2.1.2.1 Electronic Theodolites

Over the last two decades, the technological progress in angle measurements has been mainly in the automation of the readout systems of the horizontal and vertical circles of the theodolites. The optical readout systems have been replaced by various, mainly photo-electronic, scanning systems of coded circles with an automatic digital display and transfer of the readout to electronic data collectors or computers (Kahmen and Faig, 1988; Cooper, 1982). Either decimal units (gons) or traditional sexagesimal units of degrees, minutes, and seconds of arc may be selected for the readout. The sexagesimal system of angular units, which is still commonly accepted in North America (particularly in the U.S.A.), is used throughout this chapter. The relationship between the two systems is that 360° = 400 gons.

As far as accuracy is concerned, electronic theodolites have not brought any drastic improvements in comparison with precision optical theodolites. Some of the precision electronic theodolites, such as the Kern E2 (discontinued production), Leica (Wild) T2002 and T3000, and a few others, are equipped with microprocessor controlled biaxial sensors (electronic tiltmeters) which can sense the inclination (mislevelling) of the theodolite to an accuracy better than 0.5" and

automatically correct not only vertical but also horizontal direction readouts. In optical theodolites in which the inclination is controlled only by a spirit level, errors of several seconds of arc in horizontal directions could be produced when observing along steeply inclined lines of sight. Therefore, when selecting an electronic theodolite for precision surveys, one should always choose one with the biaxial levelling compensator.

Human errors of pointing the telescope to the target, centering errors, and environmental influences are the main factors limiting the achievable accuracy. The environmental influence of atmospheric refraction is a particular danger to any optical measurements. The gradient of air temperature, dT/dx, in the direction perpendicular to the line of sight is the main parameter of refraction. Assuming that a uniform temperature gradient persists over the whole length S of the line of sight, the refraction error e_{ref} of the observed direction may be approximately expressed (Blachut et al., 1979) in seconds of arc by:

$$e_{ref} = 8''(SP/T^2)dT/dx \qquad (1)$$

where P is the barometric pressure in millibars, and T is the temperature in Kelvin (T = 273.15 + t°C). If a gradient of only 0.1°C/m persists over a distance of 500 m at P = 1000 mb and t = 27°C, it will cause a directional error of 4.4", which is equivalent to a 12 mm positional error of the target. One should always avoid measurements close to any surface that may have a different temperature than the surrounding air (walls of structures or soil exposed to the sun's radiation, walls of deep tunnels, etc.). If any suspicion of a refraction influence arises, the surveys should be repeated in different environmental conditions in order to randomize its effect.

Generally, with well designed targets and proper methodology, an accuracy (standard deviation) better than 1" in angle measurements can be achieved with precision electronic theodolites if three to five sets of observations are taken in two positions (direct and reverse) of the telescope. The requirement of two positions must always be obeyed in order to eliminate errors caused by mechanical misalignment of the theodolite's axial system. This applies to both the old and most of the up-to-date theodolites even if the manufacturer claims that the errors are taken care of

automatically.

2.1.2.2 Three-dimensional coordinating systems

Two or more electronic theodolites linked to a microcomputer create a three-dimensional (3-D) coordinating (positioning) system with on-line calculations of the coordinates. The systems are used for the highest precision positioning and deformation monitoring surveys over small areas. Leica (Wild) TMS and UPM400 (Geotronics, Sweden) are examples of such systems. If standard deviations of simultaneously measured horizontal and vertical angles do not exceed 1", then positions (x, y, z) of targets at distances up to ten metres away may be determined with the standard deviations smaller than 0.05 millimetres. Usually short invar rods of known length are included in the measuring scheme to provide scale for the calculation of coordinates. Some applications are given, among others, in Wilkins et al. (1988).

2.1.2.3 Electronic Distance Measurements (EDM)

Typically, short range (a few kilometres), electro-optical EDM instruments with visible or near infrared continuous radiation are used in engineering surveys, though some long range (up to 60 km) electro-optical or microwave instruments are also available. An excellent review of various types of EDM instruments, their calibration and data reduction are given in Rueger (1990) with a list of the basic characteristics of over 200 different models.

The accuracy (standard deviation) of EDM instruments may be expressed in a general form as:

$$\sigma = (a^2 + b^2S^2)^{0.5} \qquad (2)$$

where "a" contains errors of the phase measurement and calibration errors of the so-called zero correction (additive constant of the instrument and of the reflector), while the value of "b" represents a scale error due to the aforementioned uncertainties in the determination of the refractive index and errors in the calibration of the modulation frequency. Typically, the value of "a" ranges from 3 mm to 5 mm. In the highest precision EDM instruments, such as the Kern ME5000, Geomensor CR234 (Com-Rad, U.K.), and Tellurometer MA200 (Tellumat, U.K.), a =

0.2 mm to 0.5 mm thanks to a high modulation frequency and high resolution of the phase measurements in those instruments. One of the recently developed and very recommended for engineering surveys instruments is Leica (Wild) DI2002 which offers a standard deviation of 1 mm over short distances. Over distances longer than a few hundred metres, however, the prevailing error in all EDM instruments is due to the difficulty in determining the refractive index. Therefore, all EDM measurements must be corrected for the actual refractive index of air along the measured distance. An error of 1°C or an error of 3 mb in barometric pressure causes a 1 ppm (part per million or 1 $mm \cdot km^{-1}$) error of the measured distance. An extremely careful measurement of the atmospheric conditions at several points along the optical path must be performed with well calibrated thermometers and barometers in order to achieve the 1 ppm accuracy. If the meteorological conditions are measured only at the instrument station (usual practice), then errors of a few parts per million may occur, particularly in diversified topographic conditions. In order to achieve the accuracy better than 1 ppm, one has to either measure the meteorological conditions every few hundred metres (200m - 300m) along the optical path or to use EDM instruments with a dual frequency radiation source. Only a few units of a dual frequency instrument (Terrameter LDM2 by Terra Technology) are available around the world. They are bulky and capricious in use but one may achieve with them a standard deviation of ± 0.1 mm ± 0.1 ppm. Due to a very expensive price (about $300,000) and small demand, its production has been discontinued. Research in the development of new dual frequency instruments is in progress. In deformation measurements one may reduce somewhat the influence of refraction by 'calibrating' the distance observations to the object targets by comparing the results with the 'fixed' distances between stable stations of the reference network.

Influence of relative humidity may be neglected when using common electro-optical EDM instruments in moderate climatic conditions. The negligence of humidity, however, may cause errors up to 2 ppm in extremely hot and humid conditions. Therefore, in the highest precision measurements, psychrometers with wet and dry thermometers should be used to determine the correction due to water vapour content. One should always use rigorous formula, available, for example, in Rueger (1990) to calculate the refractive index correction rather than diagrams or simplified calculation methods supplied by the manufacturers.

All EDM instruments must be frequently calibrated for the zero correction and for scale (change in the modulation frequency). The zero correction usually significantly changes with time and may also be a function of the intensity of the reflected signal. In some older EDM instruments, the zero correction may demonstrate phase dependent cyclic changes. In engineering projects of high precision, the EDM instruments should be calibrated at least twice a year, or before and after each important project, following special procedures described in (Rueger 1990) among others. The calibration must account for all combinations of EDM-reflector pairings since each reflector may also have a different additive constant correction. Additional errors, which are not included in the general error equation (2), may arise when reducing the results of the spatially measured distances to a reference plane depending on the accuracy of the reduction corrections.

Recently, a few models of EDM instruments with a short pulse transmission and direct measurement of the propagation time have become available. These instruments, having a high energy transmitted signal, may be used without reflectors to measure short distances (up to 200 m) directly to walls or natural flat surfaces with an accuracy of about 10 millimetres. Examples are the Pulsar 500 (Fennel, Germany) and the Leica (Wild) DIOR 3002.

2.1.2.4 Total Stations and Survey Robots

Any electronic theodolite linked to an EDM instrument and to a computer creates a total surveying station which allows for a simultaneous measurement of the three basic positioning parameters, distance, horizontal direction, and vertical angle, from which relative horizontal and vertical positions of the observed points can be determined directly in the field. Several manufacturers of survey equipment produce integrated total stations in which the EDM and electronic angle measurement systems are incorporated into one compact instrument with common pointing optics. Different models of total stations vary in accuracy, range, sophistication of the automatic data collection, and possibilities for on-line data processing. One of the most recommended total stations for precision engineering surveys is the Leica (Wild) TC2002 which combines the precision of the aforementioned electronic theodolite, Leica (Wild) T2002, with the precision EDM instrument, Leica (Wild) DI2002, into one instrument with a coaxial optics for

both the angle and distance measurements.

For continuous or frequent monitoring of deformations, fully automatic monitoring systems based on computerized and motorized total stations have recently been developed. The first system was Georobot (Kahmen and Suhre, 1983). The recent advanced systems include for example, the Geodimeter 140 SMS (Slope Monitoring System) and the Leica (Wild) APS and Georobot III systems based on the motorized TM 3000 series of Leica (Wild) electronic theodolites linked together with any Leica (Wild) DI series of EDM. These can be programmed for sequential self-pointing to a set of prism targets at predetermined time intervals, can measure distances and horizontal and vertical angles, and can transmit the data to the office computer via a telemetry link. Similar systems are being developed by other manufacturers of surveying equipment. The robotic systems have found many applications, particularly in monitoring high walls in open pit mining and in slope stability studies. Generally, the accuracy of direction measurements with the self-pointing computerized theodolites is worse (about 3") than the measurements with manual pointing.

2.1.3 Levelling and Trigonometric Height Measurements

The old method of geometrical levelling with horizontal lines of sight (using spirit or compensated levels) is still the most reliable and accurate, though slow, surveying method. With high magnification levelling instruments, equipped with the parallel glass plate micrometer and with invar graduated rods, a standard deviation smaller than 0.1 mm per set-up may be achieved in height difference determination as long as the balanced lines of sight do not exceed 20 metres. In levelling over long distances (with a number of instrument set-ups) with the lines of sight not exceeding 30 m, a standard deviation of 1 mm per kilometre may be achieved in flat terrain. The aforementioned influence of atmospheric refraction and earth curvature are minimized by balancing the lines of sight between the forward and backward levelling rods. A dangerous accumulation of refraction error, up to 15 mm for each 100 m difference in elevation (Angus-Leppan, 1980), may take place along moderately inclined long routes due to unequal heights of the forward and backward horizontal lines above the terrain.

The recently developed Leica (Wild) NA2000 and NA3000 digital automatic levelling systems with height and distance readout from encoded levelling rods has considerably increased the speed of levelling (by about 30%) and decreased the number of personnel needed on the survey crew. However, some users of the digital level NA3000 complain that its compensating system demonstrates systematic deviations in windy weather and, therefore, cannot be classified as a high precision level unless some improvements are introduced by the manufacturer.

High precision electronic theodolites and EDM equipment allow for the replacement of geodetic levelling with more economical trigonometric height measurements (Chrzanowski, 1989; Chrzanowski et al., 1985). Using precision electronic theodolites for vertical angle measurements and any short range EDM instrument, an accuracy better than 1 mm may be achieved in height difference determination between two targets 200 m apart. To minimize the atmospheric refraction effects, the measurements must be performed either reciprocally, with two theodolites simultaneously, or from an auxiliary station with equal distances to the two targets (similar methodology as in spirit levelling). The accuracy is practically independent of the height differences and, therefore, is especially more economical than conventional levelling in hilly terrain and in all situations where large height differences between survey stations are involved. Motorized trigonometric height traversing (reciprocal or with balanced lines of sight) with precision theodolites and with the lines of sight not exceeding 250 m can give a standard deviation smaller than 2 mm per kilometre (Chrzanowski, 1990; Chrzanowski et al., 1985). With automatic data collection and on-line processing of the measurements, daily progress of up to 15 km may be achieved independent of the terrain configuration.

The refraction error is still the major problem in further increasing the accuracy of levelling. Research in this area continues.

2.1.4 Use of Global Positioning System (GPS) in Deformation Surveys

The satellite GPS offers several advantages over conventional terrestrial methods. Intervisibility between stations is unnecessary, thus allowing greater flexibility in the selection of station

locations than in the terrestrial geodetic surveys. Measurements can be taken during night or day, under varying weather conditions, which makes GPS measurements economical. With the recent developed rapid static positioning techniques, the time for the measurements at each station is reduced to a few minutes.

Though already widely used in engineering and geoscience projects, GPS is still a new and not perfectly known technology from the point of view of its optimal use and understanding of the sources of errors. The accuracy of GPS is very often exaggerated by some authors who may not quite understand the difference between the short term precision (repeatability) and actual accuracy of GPS.

The accuracy of GPS relative positioning depends on the distribution (positional geometry) of the observed satellites and on the quality of the observations. There are several sources of errors contaminating the GPS measurements. These errors can be categorized into:

(a) signal propagation errors which include effects of tropospheric and ionospheric refraction,
(b) receiver related errors which include multipath effects, variation in the antenna phase centre, receiver noise, bias in the coordinates of the station being held fixed in the data reduction process, etc., and
(c) satellite related errors which include mainly orbital errors.

Different types of errors affect GPS relative positioning in different ways (Beutler et al., 1989; Chen, 1990; Chen and Chrzanowski, 1990; Chrzanowski et al., 1991b). Some of the errors may have a systematic effect on the measured baselines producing significant scale errors and rotations. Due to the changeable geometrical distribution of the satellites and the resulting changeable systematic effects of the observation errors, repeated GPS surveys for the purpose of monitoring deformations can also be significantly influenced (up to a few ppm) by scale and rotation errors which, if undetected, may contaminate the derived deformation parameters leading to a misinterpretation of the behaviour of the deformable body. A particular attention to the systematic influences should be paid when a GPS network is established along the shore of a large body of water and measurements are performed in a hot and humid climate.

The authors' experience with the use of GPS in various deformation studies (Chrzanowski et al., 1990c; Chrzanowski and Chen, 1992) indicate that, with the available technology (receivers) and the distribution of the satellites in 1990-1991, the accuracy of GPS relative positioning over areas of up to 50 km in diameter (typical maximum dimensions of engineering projects) can be expressed in terms of the variance of the horizontal components of the GPS baselines, over a distance S, as:

$$\sigma^2 = (3\text{mm})^2 + (10^{-6}S)^2$$

if the aforementioned systematic biases (rotations and change in scale of the network) are identified and eliminated through proper modelling at the stage of the deformation interpretation (Chrzanowski et al., 1990b). The accuracy of vertical components of the baselines are, usually, 1.5 to 2.5 times worse than the horizontal components.

The solution for the systematic parameters may be obtained by either (1) combining the GPS surveys of some baselines (of a different orientation) with terrestrial surveys of a compatible or better accuracy or (2) establishing several points outside the deformable area (fiducial stations) which would serve as a 'calibration network' or (3) combining (1) and (2). These aspects must be considered when designing GPS networks for any engineering project. In the first case, for all the terrestrial l_T observables and for GPS observables l_G one could write observation equations for epochs t_o and t_i in terms of the deformation model **Bc** (displacement function, see section 2.3) in the form (Chrzanowski et al., 1990b):

$$l_T(t_i) + v_T(t_i) = l_T(t_o) + A_T B_I c, \quad \text{for all i, and}$$

$$l_G(t_i) + v_G(t_i) = l_G(t_o) + A_G B_I c + D d\eta \quad \text{for all i,} \tag{3}$$

where $d\eta$ is the vector of changes in scale and rotation parameters between the epochs t_o and t_i, B_I is the matrix constructed by superimposing matrices **B** for all the surveyed points and all the epochs, and **A** is the design matrix relating observables to the deformation model (Chrzanowski

et al., 1986b). The elements of the vectors c and $d\eta$ are estimated using the least squares method and they are statistically tested for their significance.

The influence of systematic errors in measurements over short distances (up to a few hundred metres) is usually negligible and the horizontal components of the GPS baselines can be determined with standard deviations of 3 mm or even smaller. Recent improvements to the software for the GPS data processing allow for an almost real time determination of changes in the positions of GPS stations.

Over the past few years, the U.S. Army Corps of Engineers developed a fully automated system for high-precision deformation surveys with GPS. It was designed particularly for dam monitoring (Frodge, 1992). In the continuous deformation monitoring system (CDMS) GPS antennas are located over points to be monitored on the structures. At least two other GPS antennas must be located over reference points that are considered stable. The GPS antennas are connected to computers using a telemetry link. A prototype system used 10-channel Trimble 4000SL and Trimvec post processing software. An operator can access the on-site computer network through a remote hook-up in the office. In 1989 the system was installed at the Dworshak Dam on the Clearwater River near Orofino, Idaho. The results shown that CDMS can give accuracies of 3 mm both horizontally and vertically over a 300 m baseline (Frodge, 1992). One has to be aware, however, that although GPS does not require the intervisibility between the observing stations it requires an unobstructed view to the satellites which limits the use of GPS only to reasonably open areas. One should also remember that there might be some yet undiscovered sources of errors (e.g., effects of high voltage power lines) in GPS measurements. GPS certainly revolutionizes the geodetic surveys but still more research on its optimal use and on sources of errors in deformation surveys is needed.

2.1.5 Photogrammetric Techniques

If an object is photographed from two or more survey points of known relative positions (known coordinates) with a known relative orientation of the camera(s), relative positions of any

identifiable object points can be determined from the geometrical relationship between the intersecting optical rays which connect the image and object points. If the relative positions and orientation of the camera are unknown, some control points on the object must be first positioned using other surveying techniques. Aerial photogrammetry has been extensively used in determining ground movements in, for example, ground subsidence studies in mining areas (Faig, 1984). Brandenberger and Erez (1972), Faig (1978), and Veress and Sun (1978) give examples of applications of terrestrial photogrammetry in monitoring of engineering structures . The main advantages of using photogrammetry are: the reduced time of field work; simultaneous provision of three dimensional coordinates; and, in principle, an unlimited number of points can be monitored. The accuracy of photogrammetric point determination has been much improved in the past decade, which makes it attractive for high precision deformation measurements.

Special cameras with minimized optical and film distortions must be used in precision photogrammetry. Cameras combined with theodolites (phototheodolites), for instance the Wild P-30 model, or stereocameras (two cameras mounted on a bar of known length) have found many applications in terrestrial engineering surveys including mapping and volume determination of underground excavations (Chrzanowski et al.,1967) and profiling of tunnels (Chrzanowski and Masry, 1969). The accuracy of photogrammetric positioning with special cameras depends mainly on the accuracy of the determination of the image coordinates and the scale of the photographs. The image coordinates may, typically, be determined with an accuracy of about 10 μm, though 3 μm is achievable. The photo scale may be approximately expressed as f/s, where f is the focal length of the objective lens and s is the distance of the camera from the object. Using a camera with, for instance, f = 100 mm at a distance s = 100 m, with the accuracy of the image coordinates of 10 μm, the coordinates of the object points can be determined with the accuracy of 10 mm. Special large format cameras with long focal length are used in close range industrial applications of high precision. For instance, the model CRC-1 (Geodetic Services, Inc., U.S.A.) camera with f = 240 mm, can give sub-millimetre accuracy in 'mapping' objects up to a few tens of metres away. Recently, solid state cameras with CCD (charge couple device) sensors (Lenz, 1989) have become available for close range photogrammetry in static as well as in dynamic applications. With the new developments in CCD cameras and digital image processing

techniques, continuous monitoring with real time photogrammetry becomes possible. Further development in this area is in progress.

2.1.6 Alignment Measurements

Alignment surveys cover an extremely wide spectrum of engineering applications from the tooling industry, through measurements of amplitude of vibrations of engineering structures, to deformation monitoring of nuclear accelerometers several kilometres long. Each application may require different specialized equipment.

The methods used in practice may be classified according to the method of establishing the reference line, that is:

(a) mechanical methods in which stretched wire (steel, nylon, etc.) establishes the reference line,

(b) direct optical method (called also collimation method), in which the optical line of sight or a laser beam 'marks' the line, and

(c) diffraction method in which the reference line is created by projecting a pattern of diffraction slits.

All the above methods except mechanical are affected by atmospheric refraction, as expressed by equation (1). Therefore, in measurements requiring high accuracy, the alignment must be repeated several times in different environmental conditions.

The mechanical methods with tensioned wires as the reference lines have found many applications including dam deformation surveys. This is due to their simplicity, high accuracy, and easy adaptation to continuous monitoring of structural deformations using inductive sensors over distances up to a few hundred metres (Pelzer, 1976; Gervaise, 1974). Accuracies of 0.1 mm are achievable.

The direct optical method utilizes either an optical telescope and movable targets with

micrometric sliding devices or a collimated (projected through the telescope) laser beam and movable photocentering targets. Besides the aforementioned influence of atmospheric refraction, pointing and focusing are the main sources of error when using optical telescopes. The pointing error with properly designed targets (Blachut et al., 1979) varies from 15"/M at night in calm atmospheric conditions to 60"/M in daylight with average turbulent conditions, where M is the magnification of the telescope.

Special aligning telescopes with large magnification (up to 100x) are available from, among others, Fennel-Cassell (Germany) and Zeiss-Jena (Germany). Aligning telescopes for the tooling industry and machinery alignment are available in North America from Cubic Precision. When the optical line of sight is replaced by a collimated laser beam, then the accuracy of pointing may be considerably improved if special self-centering laser detectors, with a time integration of the laser beam energy, are used (Chrzanowski et al., 1976). The use of laser allows for automation of the alignment procedure and for continuous data acquisition. When using the laser beam directly as the reference line, however, attention must be paid to the stability of the laser cavity. A directional drift of the laser beam as high as 4"/°C may occur due to thermal effects on the laser cavity. This effect is decreased by a factor of M when projecting the laser through a telescope (Chrzanowski and Janssen, 1972).

In diffraction alignment methods, a pinhole source of monochromatic (laser) light, the center of a plate with diffraction slits, and the center of an optical or photoelectric sensor are the three basic points of the alignment line. If two of the three points are fixed in their position, then the third may be aligned by centering the reticle on the interference pattern created by the diffraction grating. It should be pointed out that movements of the laser and of its output do not influence the accuracy of this method of alignment because the laser serves only as a source of monochromatic light placed behind the pinhole and not as the reference line. Therefore, any kind of laser may be employed in this method, even the simplest and least expensive ones, as long as the output power requirements are satisfied. Various patterns of diffraction slits are used in practice. The highest accuracy and the longest range are obtained with the so-called Fresnel zone plates (Chrzanowski et al., 1976) which act as focusing lenses. For instance, rectangular Fresnel

zone plates with an electro-optical centering device were used in alignment and deformation measurements of a 3 km long nuclear accelerator (Herrmansfeldt et al., 1967) giving relative accuracy (in a vacuum) of 10^{-7} of the distance. In the open atmosphere, the thermal turbulence of air seems to have a smaller effect when using the Fresnel zone plates than in the case of direct optical alignment. The laser diffraction alignment methods have successfully been applied in monitoring both straight and curved (arch) dams (Chen, 1980) using self-centering targets with automatic data recording.

2.1.7 Measurement of Extension (Change in Distance) and Strain

2.1.7.1 Types of extensometers

Various types of instruments, mainly mechanical and electromechanical, are used to measure changes in distance in order to determine compaction or upheaval of soil, convergence of walls in engineering structures and underground excavations, strain in rocks and in man-made materials, separation between rock layers around driven tunnels, slope stability, and movements of structures with respect to the foundation rocks. Depending on its particular application, the same instrument may be named an extensometer, strainmeter, convergencemeter, or fissuremeter.

The various instruments differ from each other by the method of linking together the points between which the change in the distance is to be determined and the kind of sensor employed to measure the change. The links in most instruments are mechanical, such as wires, rods, or tubes. The sensors usually are mechanical, such as callipers or dial gauges. In order to adapt them to automatic and continuous data recording, electric transducers can be employed using, for instance, linear potentiometers, differential transformers, and self-inductance resonant circuits. In general, when choosing the kind of transducer for automatic data acquisition, one should consult with an electronics specialist on which kind would best suit the purpose of the measurements in the given environmental conditions (Dunnicliff, 1988).

One should point out that the precision EDM instruments, described in Section 2.1.1. with their accuracy of 0.3 mm over short distances, may also be used as extensometers particularly when

the distances involved are several tens of metres long.

If an extensometer is installed in the material with a homogeneous strain field, then the measured change Δl of the distance l gives directly the strain component $\varepsilon = \Delta l/l$ in the direction of the measurements. To determine the total strain tensor in a plane (two normal strains and one shearing), a minimum of three extensometers must be installed in three different directions.

2.1.7.2 Wire and Tape Extensometers

Maintaining a constant tension throughout the use of the wire or tape extensometer is very important. In some portable extensometers, the constant tensioning weight has been replaced by precision tensioning springs. One should be careful because there are several models of spring tensioned extensometers on the market which do not provide any means of tension calibration. As the spring ages, these instruments may indicate false expansion results unless they are carefully calibrated on a baseline of constant length, before and after each measuring campaign.

Among the most precise wire extensometers are the Kern Distometer (discontinued production) and the CERN Distinvar (Switzerland). Both instruments use invar wires and special constant tensioning devices which, if properly calibrated and used, can give accuracies of 0.05 mm or better in measurements of changes of distances over lengths from about 1 m to about 20 metres. Invar is a capricious alloy and must be handled very carefully to avoid sudden changes in the length of the wire. When only small changes in temperature are expected or a smaller precision (0.1 mm to 1 mm) is required, then steel wires or steel tapes are more comfortable to use.

Special high precision strainmeters of a short length (up to a few decimetres) are available for strain measurements in structural material and in homogeneous rocks. An example is a vibrating wire strain gauge available from Rocktest (formerly Irad Gage). The instrument employs a 150 mm steel wire in which the changeable resonant frequency is measured. An accuracy of one microstrain (10^{-6}) is claimed in the strain measurements which corresponds to 0.15 μm relative displacements of points over a distance of 150 mm.

2.1.7.3 *Rod, tube, and torpedo extensometers*

Steel, invar, aluminum, or fibreglass rods of various lengths, together with sensors of their movements, may be used depending on the application. Multiple point measurements in boreholes or in trenches may be made using either a parallel arrangement of rods anchored at different distances from the sensing head, or a string (in series) arrangement with intermediate sensors of the relative movements of the rods.

A typical accuracy of 0.1 mm to 0.5 mm may be achieved up to a total length of 200 m (usually in segments of 3 m to 6 m). The actual accuracy depends on the temperature corrections and on the quality of the installation of the extensometer. When installing rods in plastic conduit (usually when installing in boreholes), the friction between the rod and the conduit may significantly distort the extensometer indications if the length of the extensometer exceeds a few tens of metres. The dial indicator readout may be replaced by potentiometric or other transducers with digital readout systems. Telescopic tubes may replace rods in some simple applications, for instance, in measurements of convergence between the roof and floor of openings in underground mining.

Several models of torpedo borehole extensometers and sliding micrometers are available from different companies producing geotechnical instrumentation. For example, Extensofor (Telemac, France) consists of a 28 mm diameter torpedo 1.55 m long with an inductance sensor at each end. Reference rings on the casing are spaced within the length of the torpedo. The sensors and reference rings form the inductance oscillating circuits. The torpedo is lowered in the borehole and stopped between the successive rings recording changes in distances between the pairs of rings with a claimed accuracy of 0.1 millimetre. Boreholes up to several hundreds of metres long can be scanned.

2.1.7.4 *Interferometric measurements of linear displacements*

Various kinds of interferometers using lasers as a source of monochromatic radiation are becoming common tools in precision displacement measurements. A linear resolution of 0.01 µm, or even better, is achievable. One has to remember, however, that interferometric distance

measurements are affected by atmospheric refractivity in the same way as all EDM systems (Section 2.1.2). Therefore, even if temperature and barometric pressure corrections are applied, the practical accuracy limit is about $10^{-6}S$ (equivalent to 1 μm per metre). Thermal turbulence of air limits the range of interferometric measurements in the open atmosphere to about 60 m . The Hewlett Packard (U.S.A.) Model 5526B laser interferometer has found many industrial and laboratory applications in the measurement of small displacements and the calibration of surveying instruments.

2.1.7.5 Use of Optical Fibre Sensors

A new interesting development in the measurements of extensions and changes in crack-width has been reported by Haug et al. (1991). A fully automatic extensometer has been developed which utilizes the principle of electro-optical distance measurements within fibre optic conduits. The change in length of the fibre optic sensors are sensed electro-optically and they are computer controlled .

2.1.8 Tilt and Inclination Measurements

2.1.8.1 Methods of tilt measurements

The measurement of tilt is usually understood as the determination of a deviation from the horizontal plane, while inclination is interpreted as a deviation from the vertical. Thus the same instrument that measures tilt at a point can be called either a tiltmeter or an inclinometer, depending on the interpretation of the results.

As discussed in Section 2.1.3, geodetic levelling techniques can achieve an accuracy of 0.1 mm over a distance of 20 m, which would be equivalent to about 1.0" of angular tilt. This accuracy is more than sufficient in most engineering deformation measurements. Whenever a higher accuracy or continuous or very frequent collection of information on the tilt changes is necessary, however, various *in situ* instruments are used, such as (a) engineering tiltmeters and inclinometers; (b) suspended and inverted plumb lines; (c) hydrostatic levels. In addition, some other specialized instruments such as mercury/laser levels (Chrzanowski and Janssen, 1972) have

been developed but are not commonly used in practice and, therefore, are not reviewed in this section.

2.1.8.2 Tiltmeters and Inclinometers

There are many reasonably priced models of various liquid, electrolytic, vibrating wire, and pendulum type tiltmeters that satisfy most of the needs of engineering surveys. Particularly popular are servo-accelerometer tiltmeters with a small horizontal pendulum. They offer ruggedness, durability, and low temperature operation. The output signal (volts) is proportional to the sine of the angle of tilt. The typical output voltage range for tiltmeters is ±5 V, which corresponds to the maximum range of the tilt. Thus the angular resolution depends on the tilt range of the selected model of tiltmeter and the resolution of the voltmeter (typically 1 mV). There are many factors affecting the accuracy of tilt sensing. A temperature change produces dimensional changes of the mechanical components, changes in the viscosity of the liquid in the electrolytic tiltmeters, and of the damping oil in the pendulum tiltmeters. Drifts of tilt indications and fluctuations of the readout may also occur. Therefore, thorough testing and calibration are required even when the accuracy requirement is not very high.

Tiltmeters have a wide range of applications. A series of tiltmeters if arranged along a terrain profile may replace geodetic levelling in the determination of ground subsidence (Chrzanowski and Fisekci, 1982). Similarly, deformation profiles of tall structures may be determined by placing a series of tiltmeters at different levels of the structure (Kahmen, 1978).

In geomechanical engineering, the most popular application of tiltmeters is in slope stability studies and in monitoring embankment dams using the torpedo (scanning) type borehole inclinometers (usually the servo-accelerometer type tiltmeters). The biaxial inclinometers are used to scan boreholes drilled to the depth of an expected stable strata in the slope. By lowering the inclinometer on a cable with marked intervals and taking readings of the inclinometer at those intervals, a full profile of the borehole and its changes may be determined through repeated surveys. Usually the servo-accelerometer inclinometers are used with various ranges of inclination measurements, for instance, ±6°, ±54°, or even ±90°. If a 40 m deep borehole is measured every

50 cm with an inclinometer of only 100" accuracy, then the linear lateral displacement of the collar of the borehole could be determined with an accuracy of 2 millimetres. A fully automatic (computerized) borehole scanning inclinometer system with a telemetric data acquisition has been designed at the University of New Brunswick for monitoring slope stability at the Syncrude Canada tar sands mining operation (Chrzanowski et al., 1976)

2.1.8.3 Suspended and inverted plumb lines

Two kinds of mechanical plumbing are used in controlling the stability of vertical structures: (1) suspended plumb lines, and (2) floating plumb lines also called inverted or reversed plumb lines. Inverted plumb lines have an advantage over suspended plumb lines in the possibility of monitoring absolute displacements of structures with respect to deeply anchored points in the foundation rocks which may be considered as stable. In the case of power dams, the depth of the anchors must be 50 m or even more below the foundation in order to obtain absolute displacements of the surface points. If invar wire is used for the inverted plumb line, vertical movements of the investigated structure with respect to the bedrock can also be determined (Boyer and Hamlin, 1985). Caution must be used in installing plumb lines. If the plumb line is installed outside the dam, a vertical pipe of a proper inner diameter should be used to protect the wire from the wind (Chrzanowski et al., 1967). The main concern with floating plumb lines is to ensure verticality of the boreholes so that the wire of the plumb line has freedom of motion. The tank containing the float is generally filled with water to which some anti-freeze can be added. The volume of the float should be such as to exert sufficient tension on the wire. It should also be noted, however, that in a float tank thermal convection displacements may easily develop in consequence of thermal gradients which may affect measurements to a considerable extent. Hence in some cases, the whole tank should be thermally insulated.

Several types of recording devices that measure displacements of structural points with respect to the vertical plumb lines are produced by different companies. The simplest are mechanical or electromechanical micrometers. With these, the plumb wire can be positioned with respect to reference lines of a recording (coordinating) table to an accuracy of ±0.1 mm or better. Travelling microscopes may give the same accuracy. Automatic sensing and recording is

possible, for instance, with a Telecoordinator (Huggenberger, Switzerland) and with a Telependulum (Telemac, France). An interesting Automated Vision System has been developed by Spectron Engineering. The system uses CCD video cameras to image the plumb line with a resolution of about 3 μm over a range of 75 mm. Several plumb lines at the Glen Canyon dam and at the Monticello dam in California have used this system.

Two sources of error which may sometimes be underestimated by users are: the influence of air currents, and the spiral shape of wires; see also Appendix 2. To reduce the influence of the air pressure, the plumb-line should be protected within a pipe (e.g., a PVC tube) with openings only at the reading tables.

2.1.8.4 Optical Plummets

Several surveying instruments companies produce high precision optical plummets. Leica (Wild) ZL (zenith) and NL (nadir) plummets which offer the accuracy of 1/200,000. Both can be equipped with laser. The atmospheric refraction is the major source of errors.

2.1.8.5 Hydrostatic Levelling

If two connected containers are partially filled with a liquid, then the heights h_1 and h_2 of the liquid in the containers are related through the hydrostatic equation

$$h_1 + P_1 / (g_1 r_1) = h_2 + P_2 / (g_2 r_2) = \text{const.} \qquad (4)$$

where P is the barometric pressure, g is gravity, and r is the density of the liquid which is a function of temperature.

The above relationship has been employed in hydrostatic levelling. The ELWAAG 001 (Bayernwerke, Germany) is a fully automatic instrument with a travelling (by means of an electric stepping motor) sensor pin which closes the electric circuit upon touching the surface of the liquid.

Hydrostatic levelling is frequently used in the form of a network of permanently installed instruments filled with a liquid and connected by hose-pipes to monitor change in height differences of large structures (Robotti and Rossini, 1984). The height differences of the liquid levels are automatically recorded. The accuracy ranges from 0.1 mm to 0.01 mm over a few tens of metres depending on the types of instruments. The main factor limiting the survey accuracy is the temperature effect. To reduce this effect the instrument must either be installed in a place with small temperature variations, or the temperature along the pipes must be measured and corrections applied, or a double liquid (e.g., water and mercury) is employed to derive the correction for this effect. For the highest accuracy, water of a constant temperature is pumped into the system just before taking the readings. The instruments with direct measurement of the liquid levels are limited in the vertical range by the height of the containers. This problem may be overcome if liquid pressures are measured instead of the changes in elevation of the water levels. Pneumatic pressure cells or pressure transducer cells may be used. Numerous examples of various settlement gauges based on that principle are given in Hannah (1985). Meier (1991) describes the use of a differential pressure hydrostatic level with telemetric data acquisition in monitoring the Albigna dam in Switzerland.

2.1.9 Concluding Remarks on Monitoring Techniques

This brief review of basic monitoring techniques indicates that, from the point of view of the achievable instrumental accuracy, the distinction between geodetic and geotechnical techniques does not apply any more. With the recent technological developments in both geodetic and geotechnical instrumentation, at a cost one may achieve almost any, practically needed, instrumental resolution and precision, full automation, and virtually real-time data processing. Remotely controlled telemetric data acquisition systems, working continuously for several months without recharging the batteries in temperatures down to -40°C, are available and their cost is reasonable. Thus, the array of different types of instruments available for deformation studies has significantly broaden within the last few years. This creates a new challenge for the designers of the monitoring surveys: what instruments to choose, where to locate them, and how to combine them into one *integrated monitoring scheme in which the geodetic and*

geotechnical/structural measurements would optimally complement each other.

As far as the actual accuracy of deformation surveys is concerned, the main limiting factors are not the instrument precision but the environmental influences and ignorance of the users, namely:

- the aforementioned atmospheric refraction,
- thermal influences, affecting the mechanical, electronic, and optical components of the instruments (in any type of instrumentation) as well as the stability of survey stations,
- local instability of the observation stations (improper monumentation of survey stations and improper installation of the *in situ* instrumentation),
- lack of or improper calibration of the instruments,
- lack of understanding by the users of the sources of errors and of the proper use of the collected observations.

The problem of calibration is very often underestimated in practice not only by the users but also by the manufacturers. In long-term measurements, the instrument repeatability (precision) may be affected by aging of the electronic and mechanical components resulting in a drift of the instrument readout. Of particular concern are geotechnical instruments for which the users, in general, do not have sufficient facilities and adequate knowledge for their calibration. The permanently installed instruments are very often left *in situ* for several years without checking the quality of their performance.

The last aspect, the lack of understanding of the sources of errors affecting various types of measurements and the proper data handling is, perhaps, the most dangerous and, unfortunately, the frequent case in measurements of deformations in North America. The measurements and particularly processing of the geodetic surveys, are usually in the hands of self-proclaimed 'surveyors' at the technician level or even without any formal education. In this case, even the most technologically advanced instrumentation will not supply the expected information. There are only two universities in North America, both in Canada, the University of New Brunswick and the University of Calgary, which offer a broad specialization in engineering surveys of high precision and teach surveying engineering students how to optimally use both geodetic and

geotechnical monitoring techniques. Unfortunately, the small supply of graduates (about 30 per year) from their surveying engineering programs is much below the actual needs.

2.2 DESIGN OF MONITORING SCHEMES

2.2.1 General Design Criteria

When designing a monitoring network one has to remember that the main purpose of the monitoring surveys is:

(1) to check whether the behaviour of the investigated object and its environment follows the predicted pattern so that any unpredicted deformations could be detected at an early stage, and

(2) in the case of any abnormal behaviour, to give an account, as accurately as possible, of the actual deformation status which could be used for the determination of the causative factors which trigger the deformation.

In the first case, the design of the monitoring scheme must include stations at the points where maximum deformations have been predicted plus a few observables at the points which, depending on previous experience, could signal any potential unpredictable behaviour, particularly at the interface between the monitored structure and the surrounding material. The amount of the expected deformations may be predicted using either deterministic modelling (using, for instance, the finite or boundary element methods), or empirical (statistical) prediction models (see section 2.3). Once any abnormal deformations are noticed, additional observables have to be added at the locations which would be indicated by the preliminary analysis of the monitoring surveys as being the most sensitive for the identification of causative factors. Some redundant monitoring instruments and points are absolutely necessary for checking the reliability of the measurements, especially in some critical parts of the structure. One should bear in mind that any monitoring instruments, even if they have been installed permanently, cannot rule out defects and failures. Thus, any monitoring system should be sufficiently redundant. By redundancy one means keeping parallel but separate sets of instruments and, in addition, facilities for evaluating data by

double-checking, using alternative measurement methods. Examples would be the determination of relative displacements using alignment surveys versus displacements obtained from a geodetic monitoring network, the measurement of tilts with tiltmeters versus geodetic levelling, etc. Thus ,a properly designed monitoring scheme should have a sufficient redundancy of measurements using different measuring techniques and such geometry of the scheme that self-checking, through geometrical closures of loops of measurements, would be possible. One should stress that a *poorly designed monitoring survey is a waste of effort and money* and may lead to a dangerous misinterpretation.

The accuracy (at the 95% probability level) of the monitoring measurements should be equal to at least 0.25 of the predicted value of the maximum deformations for the given span of time between the repeated measurements. However, once any abnormal deformations are noticed, there is no limit, other than economic, for the maximum possible accuracy required. The higher the accuracy of the measurements, the easier it will be to determine the mechanism of the unpredicted deformations. Thus, the monitoring schemes may require frequent updating and upgrading of the initial design over the duration of the monitoring project.

Generally, the design of a monitoring scheme includes, among many other aspects, the following tasks:

(1) Identification of the parameters to be observed.
(2) Selection of locations for the monitoring stations (both object and reference points if applicable).
(3) Determination (pre-analysis) of the required accuracy and of the measuring range.
(4) Determination of the required frequency of repeated observations.
(5) Selection of the types of instruments and sensors to be used (various alternatives).
(6) Design of testing and calibration facilities.
(7) Design of the data management system.
(8) Preparation of a scenario for instrumentation failure (design of redundancy).
(9) Cost analysis and final decision on the selected monitoring scheme.

Since each deformable object may require different parameters to be observed, different instrumentation, different accuracies, and different frequencies of the observation, detailed specifications will significantly vary not only from one type of tobject to another, but also for the same type of object depending on the local surrounding conditions. Therefore, the brief discussion, on the first four tasks as listed above which follows, is very general and has been limited to typical conditions of concrete and emankment dams only.

2.2.2 Basic Considerations in Designing Monitoring Schemes for Large Dams

2.2.2.1 General Deformation Behaviour of Dams

Any dam is subjected to external and internal loads that cause deformation and permeability of the structure and its foundation. Deformation and seepage are clearly a function of such loads. Any sign of abnomal dam behaviour could signal a threat to dam safety.

Dam deformation patterns vary according to types of dam, foundation conditions, and external loads. Due to the differences in construction materials, the behaviour of concrete dams is completely different from that of embankment dams. In concrete dams, the deformation is mainly elastic, depending on reservoir water pressure and temperature variations. Permanent deformation may, however, be caused by the subsoil adapting to the new loads, concrete aging, or foundation rock fatigue. In such cases, the deformation is without danger as long as it does not exceed some critical value. The case of an earth dam is altogether different. Deformation here is, to a large extent, permanent. Under the impact of the self-weight of the embankment and hydrostatic pressure of the reservoir water, the fill material (and the foundation if consisting of soil) continues to settle - although at a decreasing rate - for decades after construction. In addition, permanent horizontal deformation of the embankment is due to reservoir water pressure and is mainly perpendicular to the embankment centreline. Actual elastic deformation is slight, and not typical of earth dam behaviour.

Deformation values vary considerably according to the type of dam. They are expected in millimetres and centimetres for concrete dams, and in centimetres or decimetres for embankment

dams.

Loads and the dam's response to them should be carefully monitored for any sign of abnormality as early as possible, and action promptly taken before that abnormality becomes a threat to safety. Monitoring consists of both measurements and visual inspections, neither being sufficient on their own. Every dam should be equiped with appropriate instrumentation according to dam type and size as well as to particular site conditions.

In view of the difference between concrete and embankment dam behaviour, a monitoring scheme cannot be organized in the same way for both types. In concrete dams, monitoring is essentially a matter of observing behavioural trends in both elastic and plastic deformation. The work consists of comparing measured deformation to the predicted normal behaviour, assessed through analysis or some other method. In embankment dams on the other hand, permanent deformation trends should be closely monitored for any sign of abnormality.

2.2.2.2 Identification of Parameters to be Observed in Concrete Dams

Absolute horizontal and vertical displacements. These measurements are particularly intended to determine the small displacements of points representive of the behaviour of the dam, its foundation, and abutments with respect to some stable frame. Geodetic surveys are often used for this purpose. The 'absolute values' can be obtained only if the reference points are stable. Here, the GPS technique helps in establishing stations far enough from the dam to be outside of the deformation zone of the reservoir. In order to efficiently check their stability (see Section 2.3.2.1) the number of reference points must be not less than 3, preferably 4, for vertical control, and 4, preferably 6, for horizontal, and they should be connected together by observables, with as much redundancy as possible.

Horizontal displacements in a critical direction, usually perpendicular to the axis of dam, can be surveyed with alignment techniques if the reference points of the alignment survey are stable or their movements can be determined by other techniques, for instance, by inverted plumblines with a stable anchor point, or by geodetic methods.Vertical absolute displacements can be

determined by geodetic levelling with respect to deeply anchored vertical borehole extensometers or to deep benchmarks located near the dam. Long levelling lines connecting the dam with benchmarks located several kilometres outside the deformation zone are not recommended due to the accumulation of errors.

Relative movements. Deflections (inclinations), of a dam are usually measured by direct or inverted plumblines. With reference to a horizontal line along the axis of the dam, different alignment methods are used in different levels of galleries to determine the relative movements between the blocks. Extensometers have now become important instruments for measuring differential foundation movements. A combination of geodetic levelling with suspended invar wires equiped with short reading scales at different levels of the dam and connected to borehole extensometers (Fig.1) can supply all the needed information on the relative vertical movements as well as on the absolute vertical displacements and relative tilts (Chrzanowski and Secord, 1990).

Foundation subsidence and tilts. They are measured with geodetic levelling, hydrostatic levelling, and tiltmeters. The last two are usually permanently installed in galleries.

Strain measurements. Strain gauges are preferably embedded in the concrete during construction, installed on the faces of the dam after completion, or even embedded in foundation boreholes.

Temperature. Temperature measurements should provide information on the thermal state of the concrete, water temperature at various levels, and atmospheric temperature. Temperature in the concrete is usually measured by telethermometers (thermistors, thermocouples, bi-metal thermometers) installed in the dam body.

Uplift and leakage measurements. These measurements are generally carried out by non-specific instruments. More elabrate devices may, however, be required to measure hydraulic pressure inside the rock. For leakage measurements, it is important to combine an effective drainage system with these instruments.

Joint measurements. Measurements are justified only in the case of joints separating two unsealed structures or to check grouting in dome or arch-gravity dams. Cracks are measured by the same methods, the instruments being installed on the surface.

Water level measurements. Water level in reservoir is one of the most important acting loads to a dam. For physical interpretation its measurement should coincide in time with the measurements of other deformation quantities.

2.2.2.3 Identification of Parameters to be Observed in Embankment dams

Horizontal displacements. Horizontal displacements of the crest and other important points of embankment (berms, etc.) can be measured with geodetic methods and alignment. The comments made for concrete dams are also valid here. It is also possible to detect relative horizontal displacements of points inside the embankment by means of inclinometers.

Groundwater and pore water pressures. Ground water and pore pressures are very significant in moniotoring earth dams. The pattern of seepage and pore water pressure, especially in the foundation and the impervious core, has a significant impact on the normal behaviour of embankment dams. Since pore water pressures should not exceed design values, they must be carefully monitored, possibly with pressure cells. The greater the number of measurement profiles and the number of cells per profile, the more useful the data obtained will be.

Settlements. For those occurring in accessible places, geodetic or hydrostatic levelling is customarily used to determine the settlements. The settlements of the foundation, or of interior structural parts which are not accessable (core, foundation contact), are detected through settlement gauges. The settlements of individual layers of the embankment should be monitored. This can be done through settlement gauges installed in the different layers.

Total pressure measurements. It is sometimes necessary to check the total pressure inside the embankment or between the embankment and the foundation or adjacent structures.

Water level measurements. Water level in the reservoir is the most important load on an earth dam, causing horizontal movements and seepage. Its measurement should coincide in time with measurement of the deformations and seepage.

2.2.2.3 Location of Monitoring Instruments

In addition to the general guidelines given above, for gravity dams, each block should have at least one point. Tilts of the foundation should be measured at the center for small structures, and at not less than three points for larger structures.

For multiple-arch and buttress dams, monitoring points should be located at the head and downstream toe of each buttress. In the case of massive buttresses and large arches, special attention should be paid to the foundations of the buttresses. If the buttresses are tranversed by construction joints, the behaviour of joints should be observed.

For arch-gravity dams and thick arch dams, absolute displacements of dam toe and abutments are critical. For small structures, the deformation of the central block is monitored. However, for large structures the measurement of deformations in each block is required.

For thin arch dams, crest displacements in the horizontal and vertical are required. Special attention should be given to central cantilever, abutments, and abutment rock.

2.2.2.4 Accuracy Requirements

No commomly accepted standards of accuracy requirements exist. As aforementioned, the accuracy at 95% probability should beequal to at least 0.25 of the maximum expected deformation regular behaviour, and as high as possible for a discovered irregularity.

For concrete dams, the accuracy for monitoring both horizontal and vertical displacements should be typically around 1 to 2 mm. For earth-rockfill dams, the accuracy should be about 10 mm for horizontal displacements, and 5 to 10 mm for settlements during construction; and 5 mm and 3 to 5 mm for the horizontal and vertical, respectively, in operation.

2.2.2.5 Frequency of Measurements

The frequency of measurements depends on the age of the structure and type of the monitoring system. If a fully automatic data acquisition system is used, the frequency of measurements does impose any problem because the data can be decoded at any preprogrammed time intervals without any logistic difficulties and, practically, at no difference in the cost of the monitoring process. However, in most cases, the fully automated systems are not yet commonly used and the frequency of measurements of individual observables must be carefully designed to compromise between the actual need and the cost. ICOLD (1986) gives the following general guidelines:

(1) Before and during construction, it may be useful to carry out some geodetic and piezometric measurments of the abutments.

(2) All measurements should be made before the first filling is started (initial operation). The dates of the sucessive measurements will depend on the level the water has reached in the reservoir. The closer the water is to the top level, the shorter will be the interval between the measurements. For instance, one survey should be conducted when the water reaches 1/4 of the total height; another survey when the water reaches mid-height; one survey every tenth of the total height for the third quarter; one survey every 2 m of variation for the fourth quarter. Moreover, the interval between two succesive surveys should never exceed a month until filling is completed.

(3) During the operation of the structure, measurements should be more frequent in the years immediately following the first filling when active deformation is in progress. For instance, geodetic surveys (more labour intensive) can be carried out four times a year, and other geotechnical measurements can be made once every 1 to 2 weeks.

(4) After the structure is stable, which takes usually 5 to 10 years or more, the above frequencies can be reduced by half. Not only the frequencies of measurement, but also the number of instruments read can be reduced according to what is learned during the first years of operation. A number of examples from different countries are given in Chapter 3 of this report.

2.2.3 Optimal Design of the Configuration and Accuracy of the Monitoring Schemes

The general guidelines and restrictions regarding the locations and accuracy of instrumentation, discussed above, still give room for choice (within the general guidelines) of the final positions of, at least, some observation stations and accuracy of the observations. This concerns mainly the geodetic monitoring networks, which may provide different solutions for the accuracy of the observed displacements depending on the selected configuration of the connecting surveys, location of the reference stations, and type of oservables (e.g. distances as opposed to angles, or an optimal combination of both).

The optimum design of geodetic positioning networks has been the subject of intensive investigations and publications by many authors over the past two decades. Most results have been summarized by Grafarend and Sanso (1985). The optimization of geodetic positioning networks is aimed at obtaining the optimum positions of the geodetic points with the optimum *accuracy, reliability, and economy* of the survey scheme taken as the design criteria. Design of deformation monitoring schemes is more complex and differs in many respects from the design of positioning networks. The design is aimed at obtaining optimum accuracies for the deformation parameters rather than for the coordinates of the monitoring stations using various types (geodetic and non-geodetic) observables with allowable configuration defects. The *sensitivity* of the monitoring scheme to detect deformations is introduced as more general than the accuracy design criterion. Very recently, a *separability* concept (Chen and Tang, 1992) has been added to the design criteria (separation between different possible deformation models).

There are practically two distinct optimal design methods:

- analytical and
- computer simulation ('trial and error').

Both methods usually involve an iteration process. The difference between them is that the former does not require human intervention and provides, mathematically, the optimum results, while the computer simulation method (CSM) provides acceptable results but they are not necessarily optimal. The CSM requires the experience of the designer but it can solve all the

design problems while the analytical methods have been limited to some particular solutions only.

Recently, a CSM has been developed at UNB (Tang et al., 1990) which does not require any human intervention during a fully automatic computational process from the moment of inputing the initial data through the iterative step-by-step upgrading of the design to the final design output. The development of analytical methods has been, however, the main focus of research. The formulation of a proper mathematical model for the optimal design of monitoring surveys, aiming at the detection of deformation parameters (geometrical and physical) rather than purely positioning parameters, was initially proposed by Niemeier and Rohde (1981) and subsequently by Chen et al. (1983). In the following years, although the underlying theory for the design of geodetic networks has been developed quite extensively, its full power of practical application has not been demonstrated in any real-life examples. An efficient algorithm has not existed until very recently. The major problem in this area was the inability of solving non-linear matrix equations involved in the network design. This problem has recently been solved by Kuang (1991) at the University of New Brunswick (UNB) by developing a multi-objective analytical design methodology which allows for a fully analytical, multi-objective optimal design (optimal accuracy and sensitivity, optimal reliability, and optimal economy) of integrated deformation monitoring schemes with geodetic and geotechnical instrumentation. The method allows for a simultaneous solution for the optimal configuration and accuracy of the monitoring scheme according to the given criteria and restrictions concerning the locations of some observation stations and required accuracy of the deformation parameters. More details with practical examples can be found in Kuang (1991) and Kuang et al. (1991). An example of the practical application of the multi-objective method in designing a geodetic monitoring network for a hydro-electric power station in eastern Canada is given in Chrzanowski and Kuang (1992).

2.3 ANALYSIS OF DEFORMATION SURVEYS

2.3.1 Concept of the Integrated Analysis

Even the most precise monitoring surveys will not fully serve their purpose if they are not properly evaluated and utilized in a global integrated analysis as a cooperative interdisciplinary effort. The analysis of deformation surveys includes (Chen and Chrzanowski, 1986):

- geometrical analysis which describes the geometrical status of the deformable body, its change in shape and dimensions, as well as rigid body movements (translations and rotations) of the whole deformable body with respect to a stable reference frame or of a block of the body with respect to other blocks, and
- physical interpretation which consists of: (a) a statistical (stochastic) method, which analyzes through a regression analysis the correlations between observed deformations and observed loads (external and internal causes producing the deformation), and (b) a deterministic method, which utilizes information on the loads, properties of the materials, and physical laws governing the stress-strain relationship which describes the state of internal stresses and the relationship between the causative effects (loads) and deformations.

Once the load-deformation relationship is established, the results of the physical interpretation may be used for the development of prediction models. Through a comparison of predicted deformation with the results of the geometrical analysis of the actual deformations, a better understanding of the mechanism of the deformations is achieved. On the other hand, the prediction models supply information on the expected deformation, facilitating the design of the monitoring scheme as well as the selection of the deformation model in the geometrical analysis. Figure 1 shows an idealized flowchart of the *integrated deformation analysis* (Chrzanowski et al. 1991). Thus, the expression *integrated analysis* means a determination of the deformation by combining all types of measurements, geodetic and geotechnical, even if scattered in time and space, in the simultaneous geometrical analysis of the deformation, comparing it with the prediction models, enhancing the prediction models which in turn, may be used in enhancing the monitoring scheme. The process is iteratively repeated until the mechanism of deformation is well understood and any discrepancies between the prediction models and actual deformations are properly explained.

Recently, the concept of global integration has been developed at UNB (Chrzanowski et al. 1990a: see Appendix 3) in which all three — the geometrical analysis of deformation and both methods of the physical interpretation — are combined into a simultaneous solution for all the parameters to be sought. The method still requires further elaboration, software development, and practical testing and, therefore, is not described in detail in this report. Appendix 3 gives the outline of the approach. Some other research centres, for instance, the University of Calgary (Teskey, 1986) work intensively towards the similar goal of a totally integrated analysis. The integrated deformation analysis requires a close cooperation of various specialists.

The deterministic and statistical modelling of deformations have been used in the analysis of dam deformations, at least in some countries, for many years with ENEL (1980) leading the advancement in the development of the computational procedures. As aforementioned, the geometrical analysis has been done so far in a rather primitive way with geotechnical/structural engineers analyzing separately the geotechnical observation data and surveyors taking care of the geodetic survey observations. The geotechnical analyses have usually resulted only in a graphical display of temporal trends for individual observables and the geodetic analysis would result in a plot of displacements obtained from repeated surveys which, very often, would not be even properly adjusted and analysed for the stability of the reference points. Over the past ten years ,an intensive study by the FIG working group has resulted in the development of proper methods for the analysis of geodetic surveys and has led to the development of the so-called UNB Generalized Method of the geometrical deformation analysis which can combine any type of observations (geotechnical and geodetic) into one simultaneous analysis. This report summarizes below the developed methodology for the geometrical analysis followed by brief descriptions of the statistical and deterministic methods used in modelling the load-deformation relationship and, finally, describing in more detail the concept of the hybrid physical analysis in which the statistical modelling is combined with the deterministic method.

2.3.2 Geometrical Analysis of Deformation Surveys

2.3.2.1 Identification of Unstable Reference Points

In most deformation studies, the information on absolute movements of object points with respect to some stable reference points is crucial. One problem which is frequently encountered in practice in the reference networks is the instability of the reference points. This may be caused either by wrong monumentation of the survey markers or by the points being located still too close to the deformation zone (wrong assumption in the design about the stability of the surrounding area). Any unstable reference points must be identified first before the absolute displacements of the object points are calculated. Otherwise, the calculated displacements of the object points and subsequent analysis and interpretation of the deformation of the structure may be significantly distorted. Figure 2 illustrates a situation where points A, B, C, and D are reference points and the others are object points. If point B has moved but this is not recognized and it is used with point A to identify the common datum for two survey campaigns, then all the object points and reference points C and D will show significant changes in their coordinates even when, in reality, they are stable.

Over the past two decades several methods for the analysis of reference networks have been developed in various research centers (Pelzer, 1974; van Mierlo, 1978; Koch and Fritsch, 1981; Niemeier, 1981; Chen, 1983; Heck, 1983; Gründig et al., 1985) within the activity of the aforementioned FIG Study Group. A conceptual review has been given by Chrzanowski and Chen (1986b). There are basically two schools of thought. One is based on the congruency test, and the other is based on defining a datum for the second epoch of measurements which is robust to unstable reference points. In the first case, a failure in the congruency test is followed by a search for the new congruency test which has a minimum statistic. The test statistics are calculated by removing points, one by one in turn, from the set of reference points until all the unstable points are identified. In the second case, a method has been developed at UNB which is based on a special similarity transformation which minimizes the first norm of the vector of displacements of the reference points. The method is described in Chen (1983), Chrzanowski, et al. (1986b), and Chen et al.(1990). The last two publications are attached as Appendices 4 and 5. The approach can be performed easily for one-dimensional reference networks and by an iterative weighting scheme for multi-dimensional reference networks until all the components of the displacement vectors satisfy the condition: $\sum| d_i |$ = minimum. In each solution, the weights

are iteratively changed to be $p_i = 1/d_i$. After the last iteration, the displacement vectors that exceed their error ellipses at 95% probability identify the unstable reference points. The displacements obtained from the iterative weighted transformation are, practically, datum independent, i.e. that whatever minimum constraints have been used in the least squares adjustment of the survey campaigns, the display of the transformed displacements will always be the same. Thus the obtained graphical display represents the actual deformation trend which is used later on in selecting the best fitting deformation model (see Section 2.3.2.2. below).

Several software packages for geometrical analysis have been developed, for example, DEFNAN [Chrzanowski et al., 1986b], PANDA [Niemeier and Tengen, 1988], and LOCAL [Gründig et al., 1985]. Some of them (e.g., DEFNAN) are applicable not only to the identification of unstable reference points, but to the integrated analysis of any type of deformations (to be discussed below), while others are limited to the analysis of reference geodetic networks only.

2.3.2.2 UNB Generalized Method for Geometrical Deformation Analysis

In order to be able to utilize any type of geodetic and geotechnical observations in a simultaneous deformation analysis, the UNB Generalized Method of the geometrical analysis has been developed (Chen, 1983; Chrzanowski et al., 1986b) at the University of New Brunswick within the activity of the FIG Study Group on Deformation Measurements. The method is applicable to any type of geometrical analysis, both in space and in time, including the earlier discussed detection of unstable reference points and the determination of strain components and relative rigid body motion within a deformable body. It allows utilization of different types of surveying data (conventional surveys and GPS measurements) and geotechnical/structural measurements. It can be applied to any configuration of the monitoring scheme as long as approximate coordinates of all the observation points are known. In practical applications, the approach consists of three basic processes:

- identification of deformation models;
- estimation of deformation parameters;
- diagnostic checking of the models, and final selection of the "best" model.

A brief description of the approach is given below.

The change in shape and dimensions of a 3-D deformable body is fully described if 6 strain components (3 normal and 3 shearing strains) and 3 differential rotations at every point of the body are determined. These deformation parameters can be calculated from the well-known strain-displacement relations if a displacement function representing the deformation of the object is known. Since, in practice, deformation surveys involve only discrete points, the displacement function must be approximated through some selected deformation model which fits the observed changes in coordinates (displacements), or any other types of observables, in the statistically best way. The displacement function may be determined, for example, through a polynomial approximation of the displacement field.

The displacement function can be expressed in matrix form in terms of a deformation model **Bc** as:

$$\mathbf{d}(x, y, z, t-t_o) = (u,v,w)^T = \mathbf{B}(x, y, z, t-t_o)\,\mathbf{c} \quad , \tag{5}$$

where **d** is the displacement of a point (x,y,z) at time t in respect to a reference time t_o; u,v,and w are components of the displacement function in the x-, y-, and z- directions, repsectively, **B** is the deformation matrix with its elements being some selected base functions, and **c** is the vector of unknown coefficients (deformation parameters).

For illustration, examples of typical deformation models (displacement functions) in two-dimensional analysis are given below.

(a) Single point displacement or a rigid body displacement of a group of points, say, block B (Figure 4a) with respect to block A. The deformation model is expressed in the form of the following displacement functions:

$$u_A = 0 \quad , \quad v_A = 0 \quad ; \quad u_B = a_o \ \text{ and } \ v_B = b_o \tag{6}$$

where the subscripts represent all the points in the indicated blocks.

(b) Homogeneous strain in the whole body and differential rotation (Figure 4b). The deformation model is linear and it may be expressed directly in terms of the strain components (ε_x, ε_y, ε_{xy}) and differential rotation, ϖ, as:

$$
\begin{aligned}
u &= \varepsilon_x x + \varepsilon_{xy} y - \varpi y \\
v &= \varepsilon_{xy} x + \varepsilon_y y + \varpi x
\end{aligned}
\tag{7}
$$

(c) A deformable body with one discontinuity (Figure 3c), say, between blocks A and B, and with different linear deformations in each block plus a rigid body displacement of B with respect to A. Then the deformation model is written as

$$
\begin{aligned}
u_A &= \varepsilon_{xA} x + \varepsilon_{xyA} y - \varpi_A y \\
v_A &= \varepsilon_{xyA} x + \varepsilon_{yA} y + \varpi_A x
\end{aligned}
\tag{8}
$$

and

$$
\begin{aligned}
u_B &= a_0 + \varepsilon_{xB}(x - x_0) + \varepsilon_{xyB}(y - y_0) - \varpi_B(y - y_0) \\
v_B &= b_0 + \varepsilon_{xyB}(x - x_0) + \varepsilon_{yB}(y - y_0) + \varpi_B(x - x_0)
\end{aligned}
\tag{9}
$$

where x_0, y_0 are the coordinates of any point in block B.

Usually, the actual deformation model is a combination of the above simple models or, if more complicated, it is expressed by non-linear displacement functions which require fitting of higher-order polynomials or other suitable functions. If time dependent deformation parameters are sought, then the above deformation models will contain time variables.

A vector Δl of changes in any type of observations, for instance, changes in tilts, in distances, or in observed strain, can always be expressed in terms of the displacement function. For example, the relationship between a displacement function and a change ds in the distance observed between two points i and j in two monitoring campaigns may be written as

(Chrzanowski et al.,1986a)

$$ds_{ij} = [(x_i - x_j)/s]\ u_j + [(y_i - y_j)/s)]\ v_j - [(x_j - x_i/s)]\ u_i - [(yj - yi)/s)]\ v_i \tag{10}$$

where u_j, v_j, u_i and v_i are components of the displacement function at points (x_j, y_j and x_i,y_i) respectively.

For a horizontal tiltmeter, the change $d\tau$ of tilt between two survey campaigns may be expressed in terms of the vertical component (w) of the displacement function as:

$$d\tau = (\partial w/\partial x)\ \sin\alpha + (\partial w/\partial y)\ \cos\alpha \tag{11}$$

where α is the orientation angle of the tiltmeter.

The functional relationships for any other types of observables and displacement functions are given in (Chen, 1983; Chrzanowski et al. 1986a; Chrzanowski et al. 1986b) and also in Appendix 6. In matrix form, the relationship is written as:

$$\Delta \mathbf{l} = \mathbf{A}\mathbf{B}_{\mathbf{d}}\ \mathbf{c} \tag{12}$$

where **A** is the transformation matrix (design matrix) relating the observations to the displacements of points at which the observations are made, and $\mathbf{B}_{\mathbf{d}}$ is constructed from the above matrix **B**(x, y, z, t-t_o) and related to the points included in the observables.

If redundant observations are made, the elements of the vector **c** and their variances and covariances are determined through least-squares approximation, and their statistical significance can be calculated (see Appendix 6). One tries to find the simplest possible displacement function that would fit to the observations in the statistically best way.

The search for the 'best' deformation model (displacement function) is based on either *a priori* knowledge of the expected deformations (for instance from the finite element analysis) or a qualitative analysis of the deformation trend deduced from all the observations taken together. In the case of the observables being the relative displacements obtained from geodetic surveys,

the iterative weighted transformation (see above in 2.3.2.1.) of the displacements gives the best picture of the actual deformation trend helping in the spatial trend analysis. In the case of a long series of observations taken over a prolonged period of time, plotting of individual observables versus time helps to establish the deformation trend and the deformation model in the time domain. In the analysis, one has to separate the known deformation trend from the superimposed investigated deformation. For example, in order to distinguish between the cyclic (seasonal) thermal expansion of a structure with a one-year period of oscillation and a superimposed deformation caused by other effects which are, for instance, linear in time, all the measurements can be analysed through a least-squares fitting of the cyclic function (see also section 2.4)

$$y = a_1\cos(\varpi t) + a_2\sin(\varpi t) + a_3 t + a_4 + a_5\delta(t_i) + \ldots, \tag{13}$$

to the observation data, where $\varpi = 2\pi/yr$, and a_3 is the rate of change of the observation (extension, tilt, inclination, etc.). The amplitude and phase of the sinusoid can be derived from a_1 and a_2. The constant a_4 is the y-intercept and the constants a_5, ... are possible slips (discontinuities) in the data series and $\delta(t_i)$ is the Kronecker's symbol which is equal to 1 when $t \geq t_i$ with t_i being the time of the occurance of the slip, and is equal to 0 when $t < t_i$. Examples of the temporal trend analyses of some geotechnical and geodetic long data sets are give in section 2.4 in Figures 8 and 9.

Summarizing, the geometrical deformation analysis using the UNB Generalized Method is done in four steps:

(1) The trend analysis in space and time domains and the selection of a few alternative deformation models which seem to match the trend and that make physical sense.

(2) The least-squares fitting of the model or models into the observation data and statistical testing of the models.

(3) The selection of the 'best' model that has as few coefficients as possible with as high a significance as possible (preferably all the coefficients should be significant at probabilities greater than 95%) and which gives as small a quadratic form of the residuals as possible.

(4) A graphical presentation of the displacement field and the derived strain field.

Examples of practical applications of the UNB Generalized Method of Deformation Analysis are given in Chrzanowski et al. (1991), Rohde (1990), Chrzanowski et al. (1989a), Secord (1985), Chrzanowski et al. (1983) and Chrzanowski et al. (1989b). A copy of the latter publication dealing with the generalized geometrical analysis of deformation of a hydro-electric power station is attached as Appendix 6.

The results of the geometrical analysis serve as an input into the physical interpretation and into the development of prediction models as discussed above in section 2.3.1.

2.3.3 Statistical modelling of the Load-Displacement Relationship

The statistical method establishes an empirical model of the load-deformation relationship through the regression analysis, which determines the correlations between observed deformations and observed loads (external and internal causes producing the deformation). Using this model, the forecasted deformation can be obtained from the measured causative quantities. A good agreement between the forecasts and the measurements then tell us that the deformable body behaves as in the past. Otherwise, as in the previous case, reasons should be found and the model should be refined.

Interpretation by the statistical method always requires a suitable amount of observations, both of causative quantities and of response effects. Let d(t) be the observed deformation of an object point at time t. For a concrete dam, for example, it can usually be decomposed into three components (Bonaldi et al, 1977; Fanelli, 1979; Chen and Chrzanowski, 1990):

$$d(t) = d_H(t) + d_T(t) + d_t(t) \qquad (14)$$

where $d_H(t)$, $d_T(t)$, $d_t(t)$ are the hydrostatic pressure component, thermal component, and the irreversible component due to the non-elastic behavior of the dam, respectively. The component $d_H(t)$ is a function of water level in the reservoir, and can be modelled by a simple polynomial:

$$d_H(t) = a_0 + a_1H(t) + a_2H(t)^2 + \dots + a_mH(t)^m \tag{15}$$

where H(t) is the elevation of the water in the reservoir. The component $d_T(t)$ can be modelled in various ways depending on the information on hand. If some key temperatures $T_i(t)$, (i = 1, 2, ..., k) in the dam are measured, then

$$d_T(t) = b_1T_1(t) + b_2T_2(t) + \dots + b_kT_k(t) \tag{16}$$

If air temperature is used, the response delay of concrete dams to the change in air temperature should be considered (Chen ,1988). If no temperataure is meausred, the thermal component can be modelled by a trigonometric function (Chen and Chrzanowski, 1986).

The irreversible component dr(t) may originate from a non-elastic phenomena like creep of concrete or creep of rock, etc. Its time-dependant behaviour changes from object to object. It may be modelled, for example, with an exponential function (ENEL, 1980). Chen (1988) has found that the following function is appropriate for concrete dams:

$$d_r(t) = c_1t + c_2\ln t \tag{17}$$

The coefficients a_i, b_i, c_i in equations (16), (17), and (18) are determined using the least squares regression analysis. The final model suggests the response behaviour of the different causative factors and is used for prediction purposes.

For an earth dam, the thermal effect is immaterial and the irreversible component becomes dominant.

It should be mentioned that the statistical method for physical interpretation is applicable not only to observed displacements, as discussed above, but also to other monitored quantities, such as stress, pore water pressure, tilt of the foundation, etc. The only difference is that the response function for each causative quantity may change.

2.3.4 Deterministic Modelling of the Load-Deformation Relationship

The deterministic method provides information on the expected deformation from the information on the acting forces (loads), properties of the materials, and physical laws governing the stress-strain relationship.

Deformation of an object will develop if an external force is applied to it. The external forces may be of two kinds: surface force, i.e., forces distributed over the surface of the body, and body forces, which are distributed over the volume of the body, such as gravitational forces and thermal stress. The relation between the acting forces and displacements d is discussed in many textbooks on mechanics. Let **d** be the displacement vector at a point and **f** be the acting force. They are related as

$$\mathbf{L}^T\mathbf{D}\mathbf{L}\mathbf{d} + \mathbf{f} = 0 \tag{18}$$

where **D** is the constitutive matrix of the material whose elements are functions of the material properties (e.g., Young's modulus and Poisson's ratio) and **L** is a differential operator transforming displacement to strain. If initial strain ε_0 and initial stress σ_0 exist, equation (18) becomes

$$\mathbf{L}^T\mathbf{D}\mathbf{L}\mathbf{d} + (\mathbf{L}^T\sigma_0 - \mathbf{L}^T\mathbf{D}\varepsilon_0) + \mathbf{f} = 0 \tag{19}$$

In principle, when the boundary conditions, either in the form of displacements or in the form of acting forces, are given and the body forces are prescribed, the differential equation (18) or (19) can be solved. However, direct solution may be difficult, and numerical methods such as the finite element or boundary element of finite differences methods are used. The finite element method (FEM) is most commonly method in structural and geotechnical engineering, particularly in modelling dam deformations.

The basic concept of the FEM is that the continuum of the body is replaced by an assemblage

of small elements which are connected together only at the nodal points of the elements. Within each element a displacement function (shape function) is postulated and the principle of minimum potential is applied, i.e., the difference between the work done by acting forces and the deformation energy is minimized. Therefore, the differential operator L is approximated by an linear algebraic operator. Numerous FEM software packages are available in the market ranging significantly in prices depending on their sophistication and adaptability to various types of material behaviour. One very powerful software package is FEMMA (Finite Elelement Method for Multidisciplinary Applications) developed at the University of New Brunswick (Szostak-Chrzanowski and Chrzanowski, 1991; Szostak-Chrzanowski, 1988) for 2-D and 3-D finite element elastic, visco-elastic, and heat transfer analyses of deformations. FEMMA has found many practical applications in dam deformation analyses (Chrzanowski et al.,1991), in tectonic plate movements (Szostak-Chrzanowski et al., 1992), in ground subsidence studies (Szostak-Chrzanowski and Chrzanowski, 1991) and in tunnelling deformations.

In the deterministic modelling of dam deformations, the dam and its foundation are subdivided into a finite element mesh. The thermal component dT and hydrostatic pressure component dH are calculated separately. Assuming some discrete water level in the reservoir, the corresponding displacements of the points of interest are computed. A displacement function with respect to water level is obtained by least squares fitting of a polynomial to the FEM-computed discrete displacements. Then, the displacements at any water level can be computed from the displacement function. In computation of the thermal components, the temperature distribution inside the structure should first be solved. Again, FEM could be used, based on some measured temperatures (boundary conditions). Both the coefficient of thermal diffusivity and the coefficient of expansion of concrete are required. The thermal components for the points of interest are calculated using FEM with computed temperature at each nodal point. The total deformation is the sum of these two components plus possible action of some other forces, e.g., swelling of concrete (Chrzanowski et al., 1991) due to alkali aggregate reaction which can also be modelled with FEM.

FEM is, certainly, a powerful tool in the determinsitic modelling of deformations. One has to

remember, however, that the output from the FEM analysis is only as good as the quality of the input and as good as the experience of the operator who must have a good understanding of not only the computer operation but, particularly, good knowledge in the mechanics of the deformable bodies. One should not treat FEM as a magic (black box) tool. Excellent remarks and comments on the numerical analysis of dam deformations are given in the ICOLD Bulletin No. 30a (ICOLD, 1987).

2.3.5 Hybrid Method of Deformation Analysis

As one can see from the above two sections, interpretation by statistical methods requires a large amount of observations, both of causative quantities and of response effects. Thus the method is not suitable at the early stage of dam operation when only short sets of observation data are available. In addition, some portions of the thermal and hydrostatic pressure effects may not be separated by the statistical modelling if the changes in temperature and in the elevation of water in the reservoir are strongly correlated. The deterministic method proves very advantageous in these aspects. The deterministic method is of an *a priori* (design) nature. It uses the information on geometric shape and material properties of the deformable body and acting loads to calculate deformations. However, due to many uncertainties in deterministic modelling such as an imperfect knowledge of the material properties, possibly wrong modelling of the behaviour of the material (particularly when a non-elastic behaviour takes place), and approximation in calculations, the computed displacements may depart significantly from the observed values d(t). In this case, if, for example, a suspicion is that the discrepancy is produced by uncertainties in Young's modulus of elasticity, E, and the thermal coefficient of expansion, α, the deterministic model can be enhanced by combining it with the statistical method, in the form (Chen, 1988)

$$d(t) + v(t) = x d_H(t) + y\, d_T(t) + c_1 t + c_2 \ln t \qquad (20)$$

where v(t) is the residual, $d_H(t)$ and $d_T(t)$ are the hydrostatic and thermal components, respectively, calculated from the deterministic modelling, and the last two terms take care of the possible irreversible component. The functional model for the irreversible component may vary and can

be changed by examing the residuals. The unknowns x, y, c_1, c_2 are estimated from the observations using the least squares estimation. The coefficient x is a function of Young's modulus and y is a function of the thermal expansion coefficient of concrete:

$$x = E_0/E \tag{21}$$

$$y = \alpha/\alpha_0 \tag{22}$$

where E_0 and α_0 are the values used in the deterministic modelling.

A few examples of a simplified combination of the two methods of modelling have been given in ENEL [1980]. They calibrated the constants of the material properties using the discrepancies between the measured displacements of a point at different epochs and that calculated from FEM. One must be aware, however, that if the real discrepancy comes from other effects than the incorrect values of the constants (e.g., non-elastic behaviour), the model may be significantly distorted.

Recently, as aforementioned, a concept of a global integration (Chrzanowski et al., 1990a) has been developed, where all three — the geometrical analysis of deformations and both methods of physical interpretation — are combined. Using this concept, deformation modelling and understanding of the deformation mechanism can be greatly enhanced. Appendix 7 gives a formulation of the concept.

2.4 AUTOMATED DATA MANAGEMENT OF DEFORMATION SURVEYS

2.4.1 Advantages and Limitations of Automation

In the total effort of deformation monitoring, the quality of the analysis of the behaviour of the object being monitored depends on the location, frequency, type, and reliability of the data gathered. The data concerned is any geotechnical observable as well as any conventional geodetic observable (angle, distance, or height difference). Apart from the location and type of instrumentation, the frequency and reliability of the data can be enhanced by employing an

"automatic" system of data gathering or acquisition and processing (including the deformation analysis). A data management system encompasses everything that happens to the data from the instant at which it is sensed to the time of analysis. Under ordinary circumstances, the interval of time between sensing and analysis may extend over several days or more. Under critical conditions, this may have to be nearly instantaneous in order to provide a warning, if necessary. The volume of data may consist of only several items (in the simplest routine investigation) to many hundreds or thousands (in very complex, critical situations, particularly if vibration behaviour is of interest). The rate of sampling may be annually, monthly, weekly, daily, hourly, or even more frequently. The amount of human involvement may range from total (a "manual" system) to virtually none (an "automatic" system). Neither extreme is practical. A manual system is labour intensive and liable to errors or blunders and is less flexible in the re-examination of data. An automatic system is attractive but has some limitations. Although a "data acquisition system" strictly involves the gathering of data, the phrase has been used by many to mean the whole system of data management. Dunnicliff (1988) has weighed the advantages and limitations of an automatic data acquisition system and they are summarized in the following two lists.

The advantages of an automatic data acquisition system are:

(1) personnel costs for reading instruments and analyzing data are reduced,
(2) more frequent readings are possible,
(3) retrieval of data from remote or inaccessible locations is possible,
(4) instantaneous transmission of data over long distances is possible,
(5) increased reading sensitivity and accuracy can be achieved,
(6) increased flexibility in selecting required data can be provided,
(7) measurement of rapid fluctuations, pulsations, and vibrations is possible,
(8) recording errors are fewer and immediately recognizable, and
(9) data can be stored electronically in a format suitable for direct computer analysis.

The limitations of an automatic system are:

(1) a knowledgeable observer is replaced by hardware, i.e., less frequent "intelligent" visual inspections,

(2) an excess of data could be generated, leading to a failure in timely response,
(3) the data may be blindly accepted, possibly leading to a wrong conclusion,
(4) there could be a high initial cost and, possibly, a high maintenance cost,
(5) often requires site-specific or custom components that may be initially unproven,
(6) complexity may require an initial stage of debugging,
(7) specialized personnel may be required for regular field checks and maintenance,
(8) a manual method is required as an alternative (backup),
(9) a reliable and continuous source of power is required, and
(10) the system may be susceptible to damage by weather or construction activity.

With an appropriate compromise between manual and automatic functions, a properly designed and working system can readily minimize the effects of the limitations mentioned above. Therefore, the advantages of an automatic (really "semi-automatic") system easily outweigh its disadvantages.

2.4.2 Examples of Automated Systems

In a variety of levels of sophistication, automated systems have become commonplace, particularly in the monitoring of dams and hydro-electric power generating stations. Italy's ENEL (Ente Nazionale per l'Energia Elettrica) and ISMES (Istituto Sperimentale Modelli e Strutture) have been leaders in the philosphy (Fanelli, 1979) and creation of monitoring sytems (Anesa et al., 1981; Bonaldi et al., 1977; Bonaldi et al., 1980; Bonaldi et al., 1985). Now, systems are well established in other countries (ICOLD, 1982), in particular: Austria (Hautzenberg, 1979; Ludescher, 1985); Canada (Cartier and Hamelin, 1988; Chrzanowski and Secord, 1990; Secord, 1990); Japan (Japanese National Committee on Large Dams, 1987); People's Republic of China (Chen and Du, 1990); Poland (Jankowski et al., 1985); Portugal (Florentino et al., 1985; Silva Gomes, 1982); Switzerland (Swiss National Committee on Large Dams, 1985; Gilg et al., 1985); and United States (Bartholomew and Haverland, 1987; Bartholomew et al., 1987; Lytle, 1982; Lytle, 1985; Walz, 1989).

Systems have also been developed for other applications, especially regarding high precision

metrology measurements (Freidsam et al., 1987; Ruland and Ruland, 1988); however, they are restricted to handling only conventional geodetic observables.

Any mention of data management systems in the literature has been usually by way of figures rather than any extensive verbal description and the emphasis has been on showing the communication aspects of a system. There are four systems which warrant mention here specifically. One, used at ENEL (Bonaldi et al., 1977), is shown in Figure 4. A second, used by the U.S. Bureau of Reclamation [USBR] (Bartholomew et al., 1987), is shown in Figure 5. Thirdly, activity in the U.S. Army Corps of Engineers [USACE] has been presented by Lytle (1982, 1985) and Walz (1989), no figure from which shows details differently than in the other three systems. Finally, Figure 6 shows a suggestion by UNB now used by N.B. Power (Secord, 1990). Silva Gomes (1982) describes efforts at LNEC (Laboratório Nacional de Engenharia Civil, Portugal) which are very similar to these four systems. A brief description of the systems is given below. Since current technology offers considerably more capacity and convenience than even five years ago, the technological aspects of ENEL and the Bureau should be kept in context. It would appear that the current trend is toward microcomputers for on-site analysis and communication to a central office is fairly routine.

2.4.2.1 The ENEL System

In the ENEL system (Figure 4), there are two major subsytems: "Off-line" which serves as a central storage of all data; and "On-line" in which most of the activity takes place. It is the On-line portion that is of interest and will be described here. Two mini-computers are involved and are linked for the teletransmission of data. The remote virtually duplicates the functions of the local, except for the actual capture of data. The local, "ESSDI/L", provides data acquisition, validation, processing, storage, and transmission to other sites. Also, it issues a warning if the observed effect differs from the expected effect (as derived from deterministic modelling) by more than an established tolerance.

2.4.2.2 The USBR System

Figure 5 illustrates the arrangement at the Calamus [embankment] Dam managed by the U.S.

Bureau of Reclamation. The main aspect of this system is the means of communication by telephone line or satellite link from the dam to various locations in the country. Two appendices to Bartholomew et al. (1987) describe, in detail, the data processing and automation of the Bureau's embankment dams. Bartholomew and Haverland (1987) discusses concrete dams but more detail, with respect to data management, is given in Bartholomew et al. (1987).

2.4.2.3 The USACE System

Lytle (1982) describes a system used by the St. Louis district, with respect to automated acquisition, processing, and plotting of data. An example of typical dialogue encountered in using the mini-computer based system is given. Walz (1989) discusses a system under development, initially to deal with totally automating piezometers in embankment dams, particularly with respect to local and district communication. The concepts of both authors are encompassed by the other three systems.

2.4.2.4 The UNB System

A data management sytem was devised by UNB for use on a IBM PC AT compatible on-site microcomputer and the essence of this system is shown in Figure 6. It was created to replace a manual system already in use for several years. In the field, a programmed data collector provides for direct connection to (and sometimes control of) instrumentation and for the keyboard entry for other equipment. The system can also accomodate manually recorded data or data directly acquired from instrumentation. The raw data are contained in observation files, archived for security, and are processed or "reduced" (using calibration, test values, etc.) into data files which are then used by various analysis and display software.

In the field, there is a check file that is either accessed during data collecting or available in hardcopy. The check file contains expected values predicted from stochastic (statistical) analyses of the data files and thus provides for a warning in the field. A warning is also given in the processing if the currently processed value differs from the most recent value in the data file, beyond a set tolerance.

The major advantage of the UNB system is that any data or derived data, whether geotechnical or geodetic (so long as it has been repeated in a suitable time series), can be brought together in the integrated deformation analysis of a structure. In the process, a time series is analysed for trend, with the separation of seasonal and long term behaviour. In the absence of actual temperature information, the trend ("y", the change in the value of an observed or derived quantity) against time ("t", in years) is described by the following equation.

$$y = a_1 \sin\hat{\omega}t + a_2 \cos\hat{\omega}t + a_3 t + a_4 + a_5 + \ldots$$

in which

$\hat{\omega}$ is 2π since a period of 1 year is assumed,

a_3 the "rate" or long term trend,

a_5 .. possible values of slips accounting for discontinuities in the data series (a_4 is also a slip, but it is required so that the fitting is not unduly constrained).

and from which the amplitude and phase can be derived to provide a comparison of seasonal behaviour among the various measurement points in the structure. Done rigourously using the method of least squares, the fitting provides a full statistical analysis of the trend with the detection of outlying or erroneous data. Figure 7 shows an example of a continuous data series and its fitting. In comparison, Figure 8 shows another series in which several interruptions in the continuity of the series occurred (e.g., change in the tape of an extensometer after damage and without regular calibration). It is possible to derive a new series from two original series or to create a series from repeated geodetic campaigns (e.g., tilt derived from levelling, Figure 9). The system can also show several series of data simultaneously, without fitting, to provide a graphical comparison of the series (Figure 10).

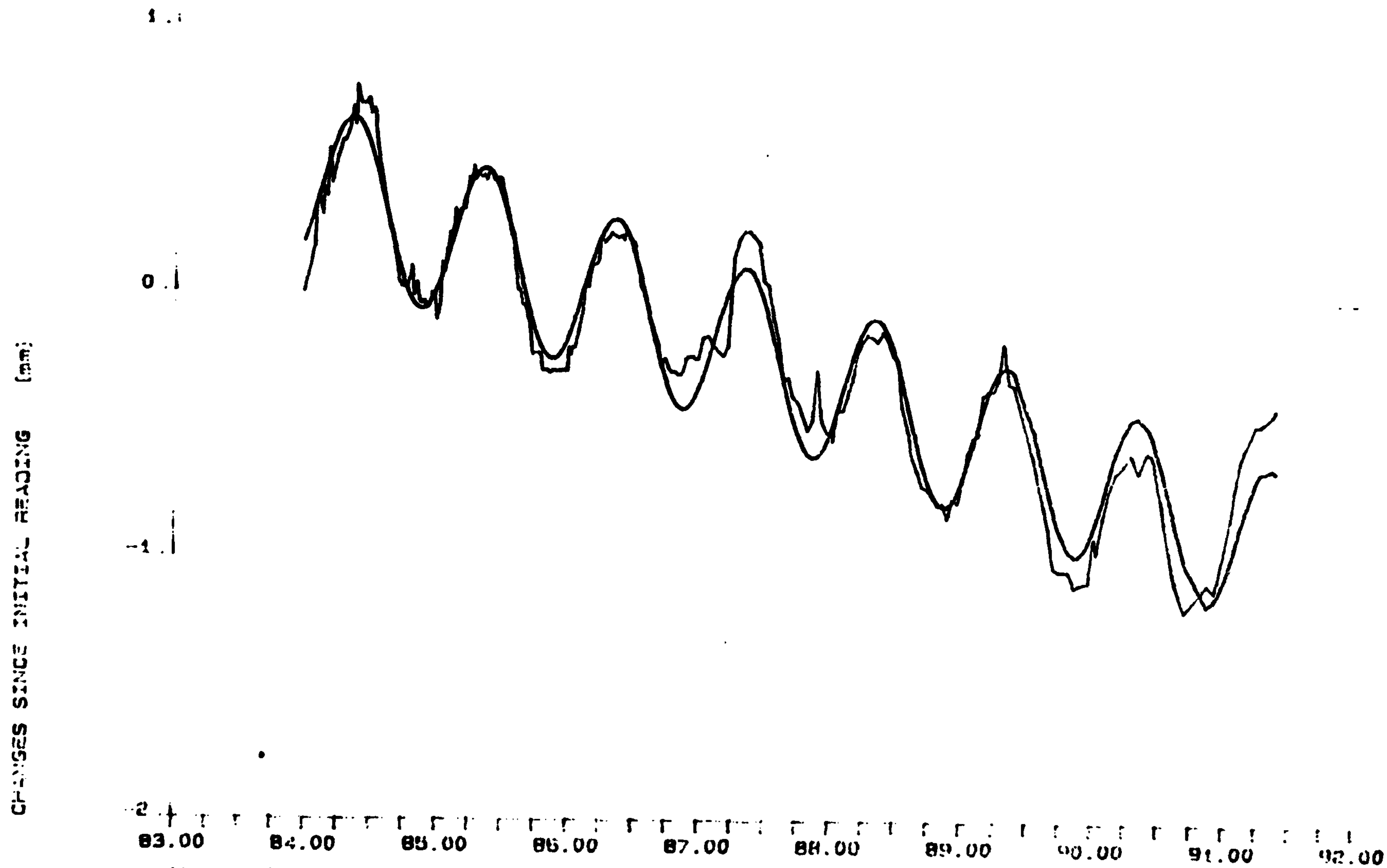
The treatment of the data can be described with reference to Figure 6. The geodetic data is treated traditionally in campaigns for adjustment ("C.A.") and spatial trend analysis ("S.T.A.", e.g., see section 2.3). Once the observations have been repeated a sufficient number of times, they can be treated as a time series ("T.S.A" time series analysis and plot, e.g., tilt from levelling, Figure 9; "S.S.A." spatial series analysis and plot, e.g., subsidence). Geotechnical series are treated in a similar mannner ("T.S.P." time series analysis and plot, e.g., Figures 7 and 8; "S.S.P." spatial series analysis and plot, e.g., borehole profile changes). The time series analyses ("T.S.A."

and "T.S.P.") are fitted with the sinusoid in the above equation to separate seasonal effects from the long term trend. All of the trend analyses are automated by command files which are setup to control fitting and automated plotting of several series in succession. All of the data can be used together in simultaneous integrated geometrical analyses following the UNB Generalized Method (e.g., "D.M.", see section 2.3). If desired, several series can be plotted simultaneously, without fitting ("Other", e.g., Figure 10). Since both the observation files and the data files are ASCII text files, they are accessible also through any text editor for manual entry or editing and can be input to other applications ("Appl'n").

2.4.3 Desirable Characteristics of an Automated System

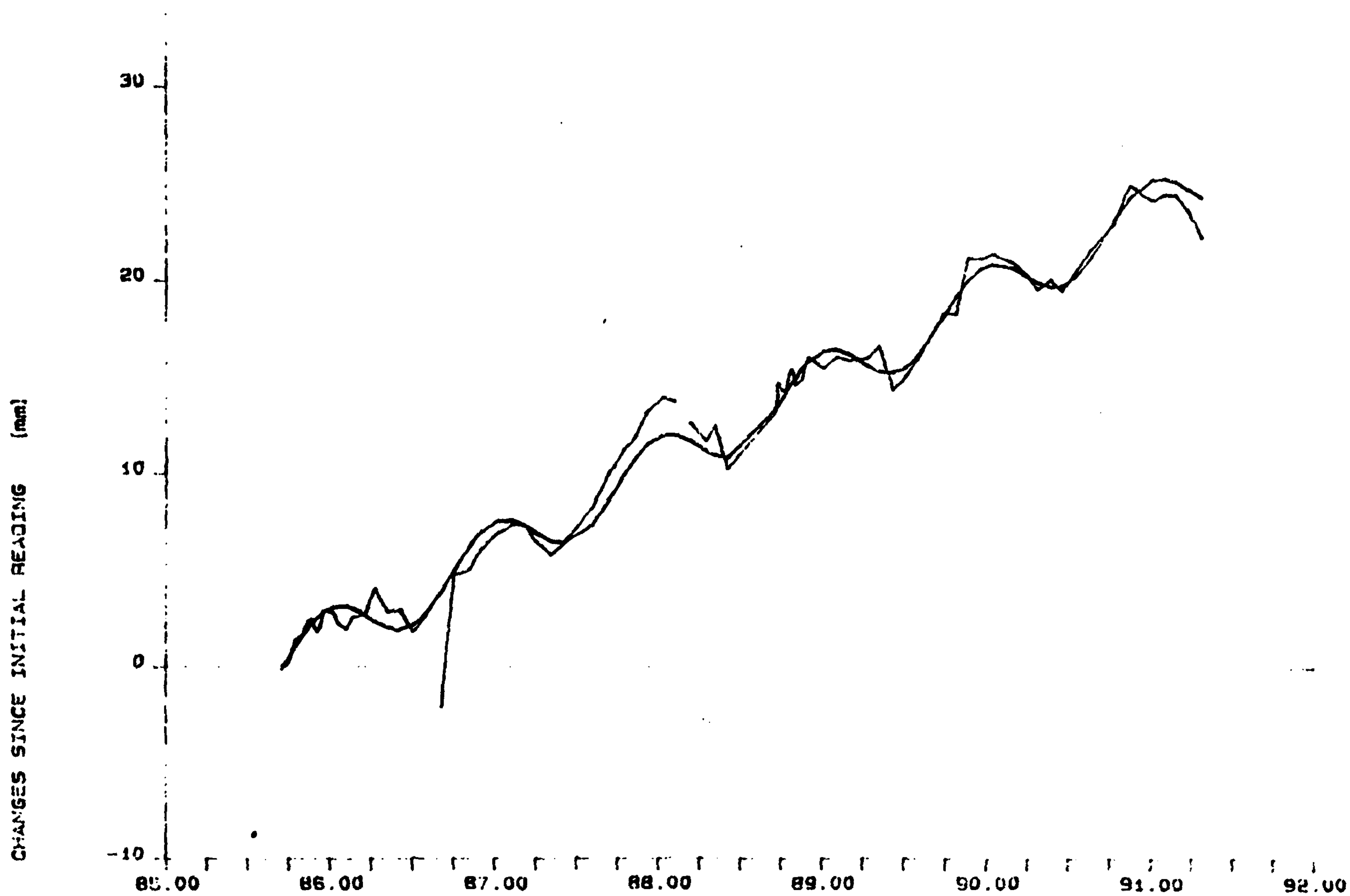
Together, these examples show the desireable characteristics of a data management system for deformation surveys (including both geotechnical and geodetic observables).

(1) Data integrity (offering checks in the field and later processing).
(2) Data security (automatic archiving and regular data file backup).
(3) Automation of acquisition, processing, and analysis.
(4) Compatibility and integration with other observables.
(5) Flexibility in access to the data for possible manual entry and editing.
(6) Data openness (useable by other software).
(7) Flexibility in the system to be easily modified to accomodate additional instrumentation or other forms of analysis.
(8) On-site immediate access to data or any of the forms of analysis.
(9) Near-real time results of trend or other analyses.
(10) Testing and calibration is an integral component of the system.



Observed Estimated, Rate (a3) -0.1840 +/- 0.0030, Extension -0.0408
Amplitude 0.3125 +/- 0.0088, Maximum at yy 5 8

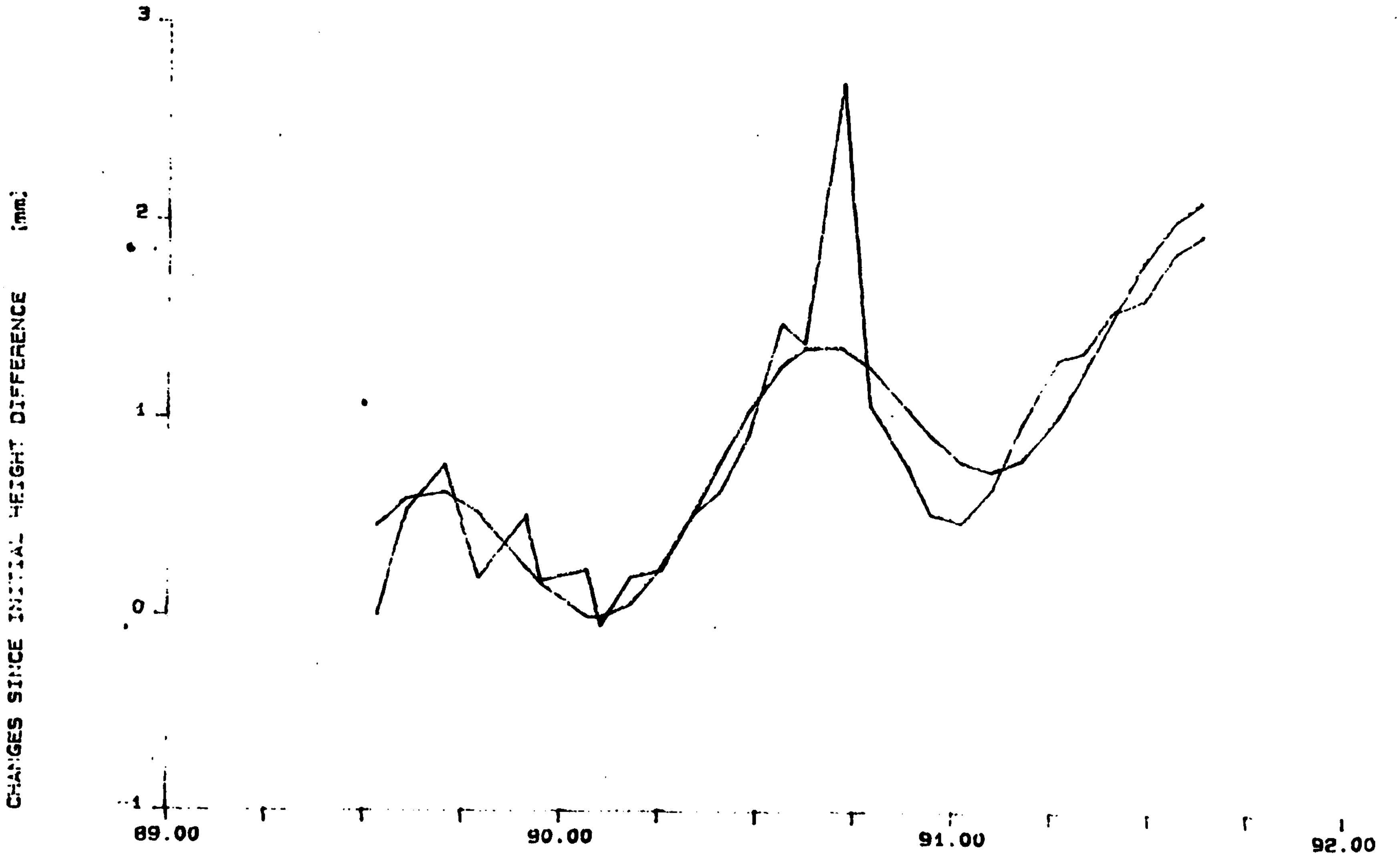
Figure 7 Extensometer BA01 collar to anchor 2 10.330 m (Alum.)
minus Extensometer BA01 collar to anchor 1 5.820 m (Alum.)
1992 9 22 input from BA01S col 2 minus BA01S col 1.



Observed Estimated, Rate (a3) 4.4984 +/- 0.1752, Extension 0.1824
Amplitude 1.5801 +/- 0.1846, Maximum at yy 12 28

Figure 8 TA4 to TA5 24.660 m calibration after 1988 02 10

1992 9 22 input from TA04-05S col 1



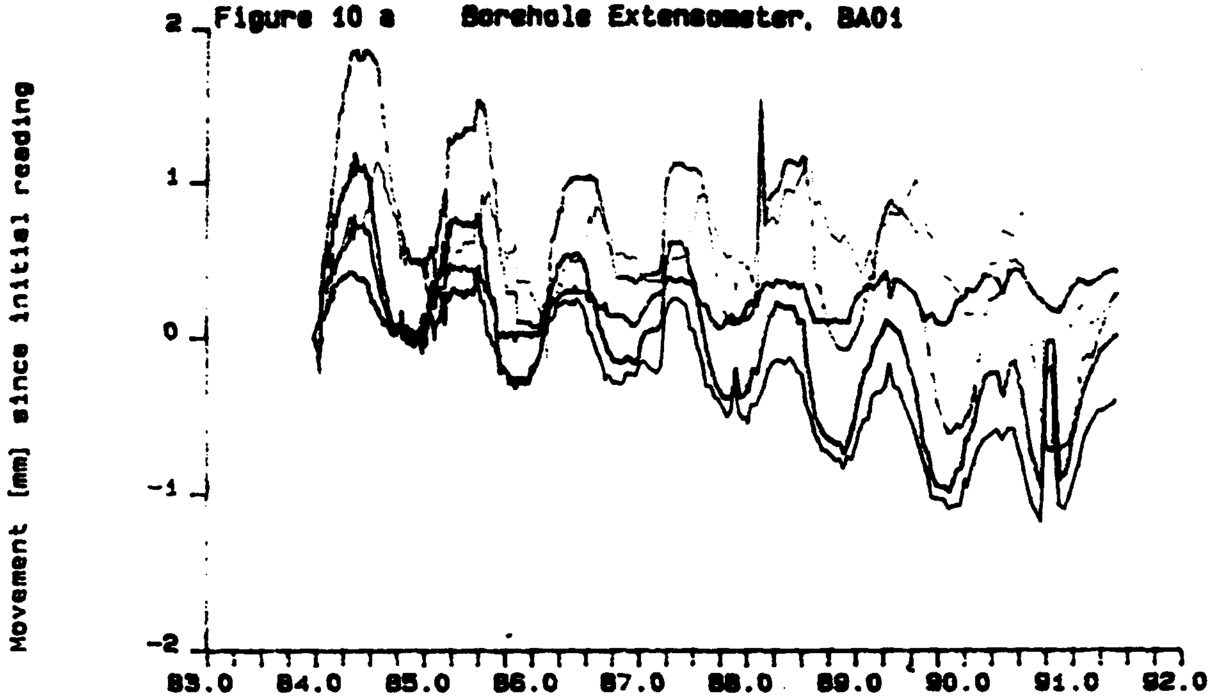
Observed Estimated. Rate (a3) 0.7537 +/- 0.1113. Tilt [mm/m] 0.0431 over 17.508 m

Amplitude 0.4986 +/- 0.0928. Maximum at yy 8 18

Figure 9 Powerhouse -29 Gallery 29 1 to 29 2

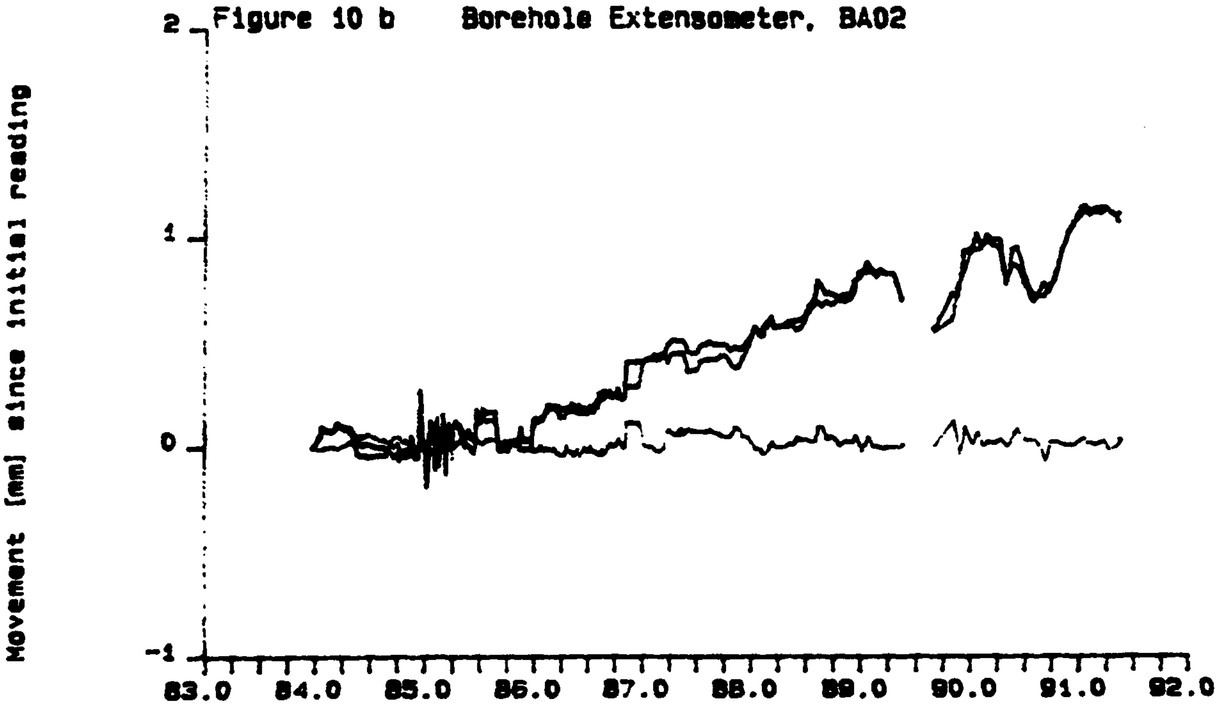
1992 9 22 output file : nut .PRT

Figure 10 a   Borehole Extensometer, BA01



Extensometer BA01 collar to anchor 1 5.820 m (Alum.)
Extensometer BA01 collar to anchor 2 10.330 m (Alum.)
Extensometer BA01 collar to anchor 3 24.840 m (Alum.)
Extensometer BA01 anchor 1 to anchor 2 (Alum.)
Extensometer BA01 anchor 2 to anchor 3 (Alum.)

Figure 10 b   Borehole Extensometer, BA02



Extensometer BA02 collar to anchor 1 10.340 m (Alum.)
Extensometer BA02 collar to anchor 2 24.330 m (Alum.)
Extensometer BA02 anchor 1 to anchor 2 (Alum.)

1992 9 23

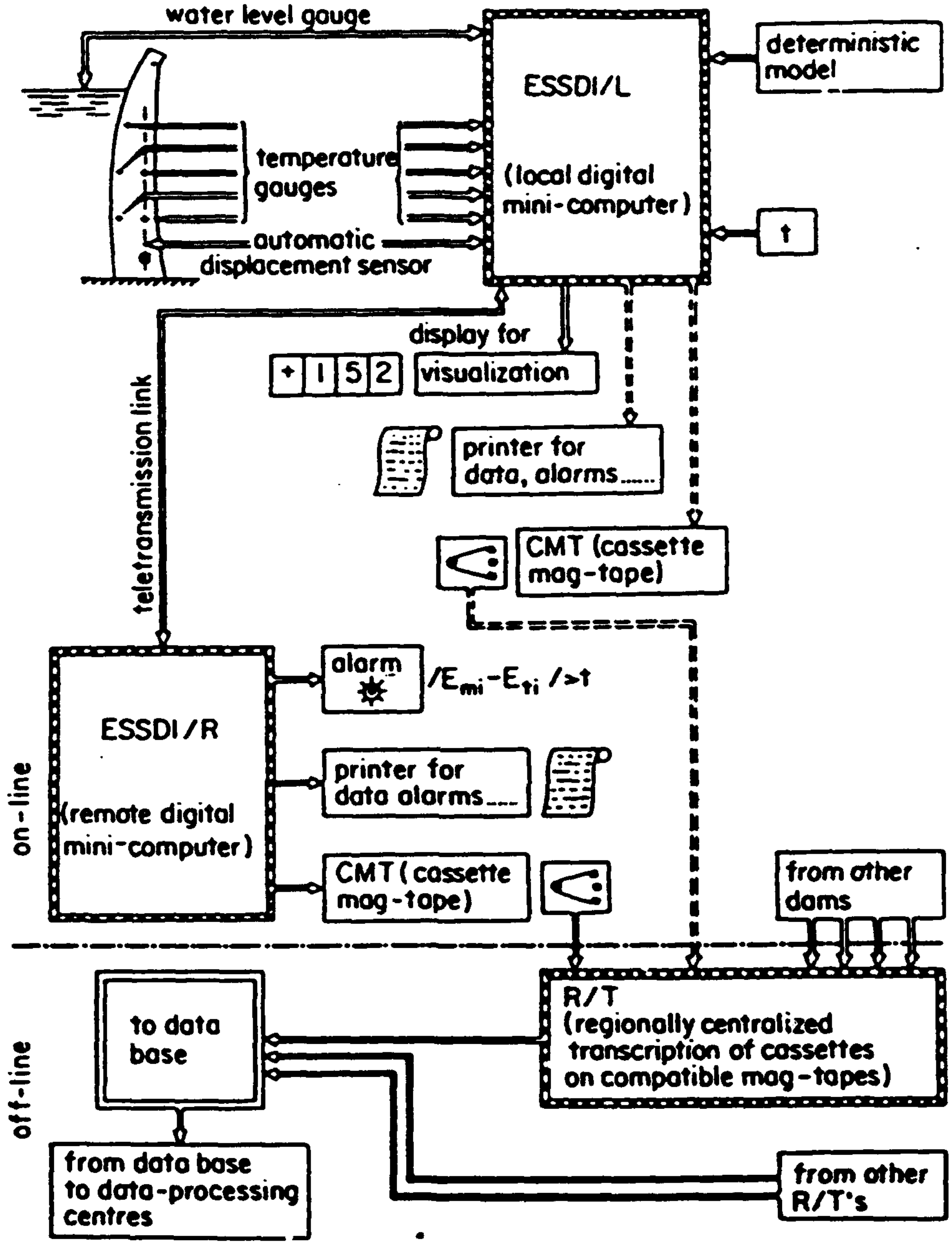


Figure 4

ENEL's hardware and schematic information flow for dam displacement control

(Bonaldi et al., 1977)

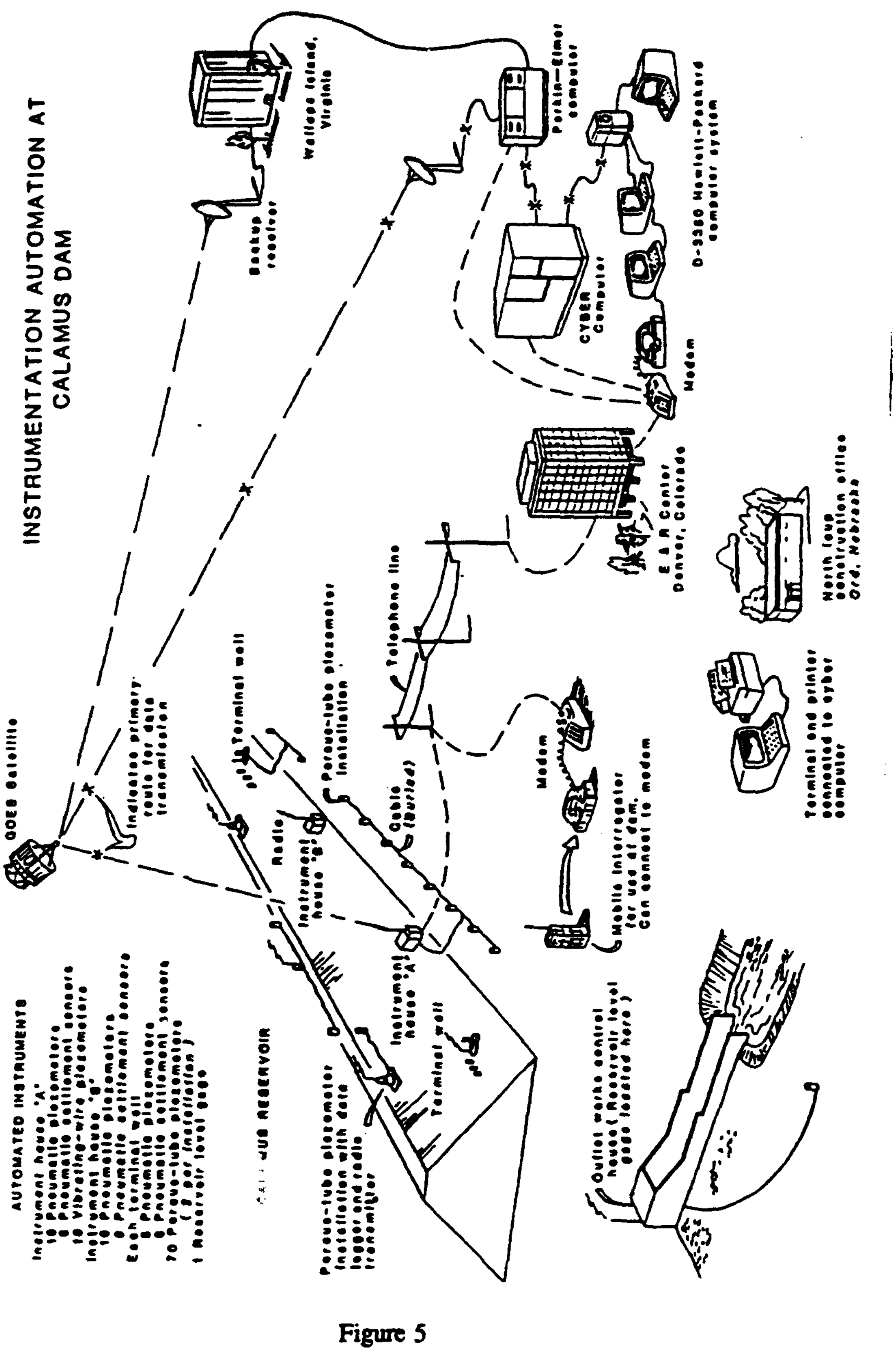
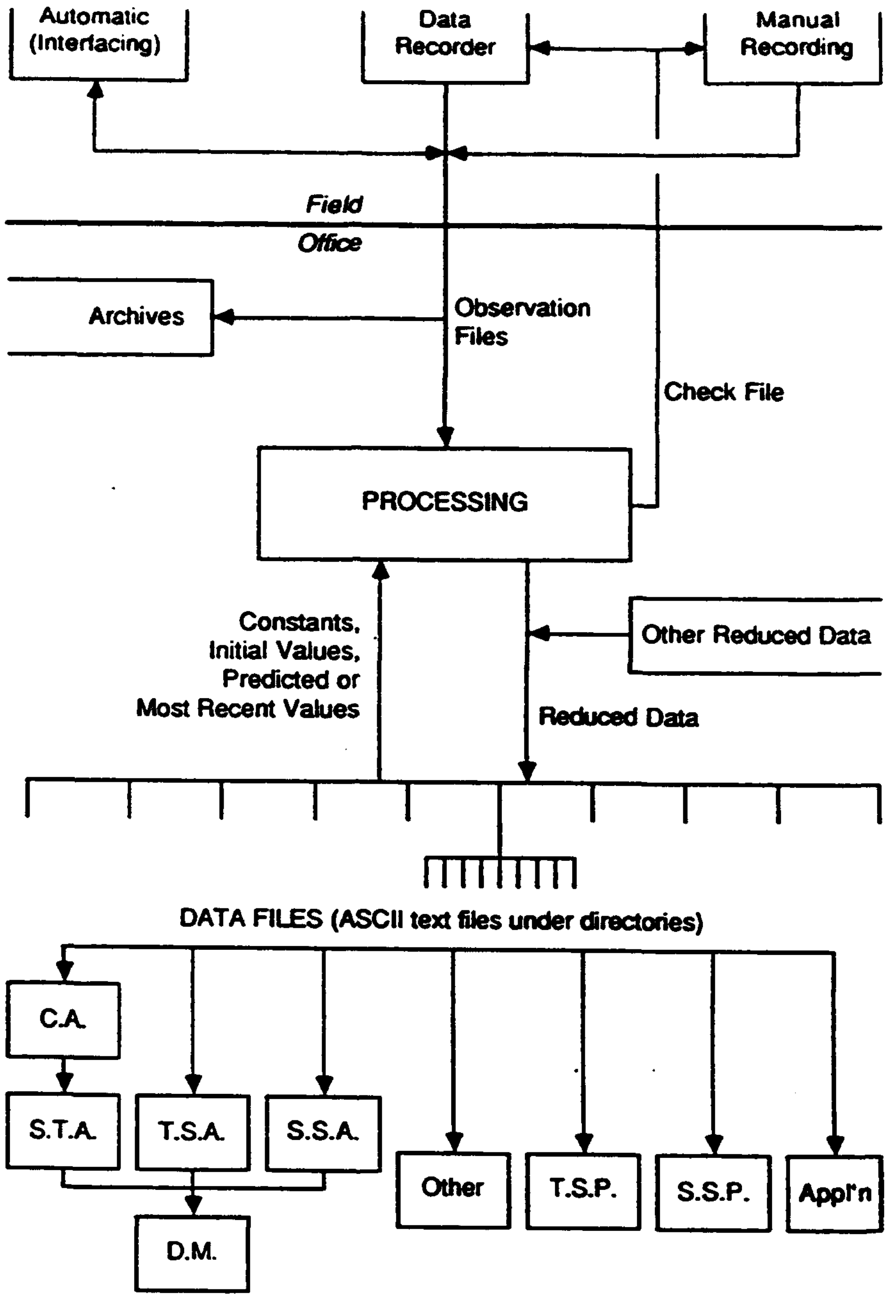


Figure 5

U.S. Bureau of Reclamation example of instrumentation automation

(Bartholomew et al., 1987)



C.A. Geodetic Campaign Adjustment
S.T.A. Spatial Trend Analysis and Plot
T.S.A. Time Series Analysis and Plot
S.S.A. Spatial Series Analysis and Plot
D.M. Integrated Geometrical Deformation Analysis (Modelling)
T.S.P. Time Series Analysis and Plot
S.S.P. Spatial Series Analysis and Plot
Other Other and future programs of the system
Appl'n other Applications (software) using the data

Figure 6

The UNB data management system for deformation analysis

(after Secord, 1990)

3. WORLD WIDE STATUS OF MONITORING AND ANALYSIS OF DAM DEFORMATIONS

3.1 GENERAL

According to the *World Register of Dams* (1988) there were a total of 36,235 large dams ($h > 15$ m) in operation around the world in 1988. This includes 29,974 embankment (earth- and/or rock-fill) dams, 4,180 gravity and 1,592 arch dams. It is worth noting that about half of the total number of large dams are in China. USA with a total of 5,469 large dams (see 1.3) is in second place and Japan in third with 2,228. Table 3.0 lists the number of dams broken down into the major types which are owned by the top ten International Commission of Large Dams (ICOLD) member countries. Between 1951 and 1986, an average of 337 dams were being constructed per year, excluding China. In China the average rate was 523 dams per year. The former Soviet Union, instead of listing 132 dams, it should probably account for 2,000 or 3,000 dams once those built by the Ministry of Agriculture and local authorities are added. This would rank the Soviet Union third behind China and USA.

There are a total of 79 countries registered with ICOLD. In order to obtain information on the procedures used in monitoring and analysis of dam deformations, a questionnaire (see 1.3) was forwarded to representative of all the ICOLD member countries.

To date, 28 countries have responded: Argentina, Australia, Austria, Bangladesh, Brazil, Canada, China, Cyprus, Czechoslovakia, France, Germany, Greece, Hungary, Ireland, Italy, Japan, Korea, Netherlands, New Zealand, Norway, Portugal, South Africa, Spain, Switzerland, Thailand, United Kingdom (UK), United States of America (USA) and Uruguay. On the basis of the questionnaire and the information gathered from ICOLD Bulletins, Proceedings of ICOLD International Congresses and other relevant publications, this reports summarizes the collected information from a few selected countries which are considered to be the leading contributors to the new

developments in deformation monitoring. A special attention has been given to USA and Canada. Note that the order in which the countries are reviewed is strictly in alphabetical order.

Table 3.0
Number of Dam Types by Country
(after *World Register of Dams*, [1988, pp.19-21])

ICOLD Member Countries	Dam Types				
	Embankment	Concrete/Masonry			Total
		Gravity	Arch & Multiple Arch	Buttress	
1. China	17,473	539	785	23	18,820
2. USA	4,694	537	192	36	5,459*
3. Japan	1,484	674	52	18	2,228
4. India	998	138	1	-	1,137
5. Spain	151	515	47	24	737#
6. Korea (Rep of)	675	15	-	-	690
7. Canada	387	195	9	17	608
8. UK	413	91	17	14	535
9. Brazil	391	107	8	10	516
10. Mexico	343	144	11	5	503
Remaining 69 Countries	2,965	1,225	611	201	5,002
TOTAL (%)	29,974 (82.72)	6,261 (17.28)			36,235

Note: * According to the USCOLD Register of Dams currently there are 5,469 dams in the USA (Sharma, 1992).
As of December 1991 Spain registered 1,031 dams (Yagüe, 1992).

3.2 ARGENTINA

Argentina has 98 large dams officially registered, some of which are monitored using solely geodetic instrumentation or both geodetic/geotechnical instrumentation (Aisiks, 1992).

Argentina has no national standards for dam monitoring. There is, however, a legislation stating that each of its 25 provinces is responsible for all dams under their jurisdiction. The current privatization of the majority of the federally owned state companies will force the State Commissions to enforce supervision and monitoring of dams by the new owners. Currently, dams owned by state organizations such as Agua y Energia Electrica S.E. and Hidronor S.A. have formulated their own specifications. These specifications are based on the international standards set out by the ICOLD bulletins and other publications.

The standard practice is to duplicate the instruments at critical points in the monitoring system or by observing the same point using two distinct types of instruments based on different principles (e.g., plumb line & traverse, and settlement gauge & levelling). Other normal practices adopted by Argentina's dam owners is to use automated data acquisition systems and seismic instrumentations in the monitoring scheme. Surveillance of dams is conducted daily by a full time staff observing and detecting any discrepancies from the regular behaviour (Naum and Aguilera, 1982).

The weakness in the monitoring system lies with the geodetic survey systems and the methods used in analysing the data. For example, in the Alicura Dam, in the Northwestern Argentina, the geodetic network is not accurate enough to detect small displacements in the most critical areas of the dam. Instead, the data from the extensometer, pendulums and load cells are used to detect these small changes (Pujol and Andersson, 1985). For the deformation analysis, the geodetic and geotechnical data are analyzed independently. Furthermore, the geotechnical data is separated and analyzed using different numerical models. Stress-strain Models are used to analyze data from load cells and strain gauges and Seepage Models are used to analyze data from piezometers (Botta et al, 1985). There is no one common method of analysis that combines the data from geodetic and geotechnical instrumentations into one integrated monitoring scheme to determine the overall behaviour of the structure.

A typical monitoring scheme used for deformations in Argentina is summarized in Table 3.1. It lists the types, numbers and location of the instruments installed and the parameters measured

within the Alicura Dam (Pujol and Andersson, 1985). The reader is referred to Botta et al (1985), Pujol and Andersson (1985) and Naum and Aguilera (1982) for a more comprehensive description on the behaviour and the installation of the instruments and the numerical models applied to the Alicura Dam.

The above information is the most current information found on dam monitoring in Argentina. According to Mr. E.G. Aisiks, Chairman of the Argentina National Committee on Large Dams, the references provided give a good representation of deformation monitoring of dams in Argentina (Aisiks, 1992).

Table 3.1
List of Instruments Installed in the Alicura Dam, Argentina
(after Pujol and Andersson [1985, pp. 394-397])

Measured Parameters	Instrument Type	Instrument Location	Qty.
Water Pressures (pore-water & interstitial pressures)	Electrical piezometers, vibrating wire	Foundation Core Filters, shells Left bank	60* 49* 7* 40*
	Pneumatic piezometers	Core Filter, shells	22 13
	Open piezometers Casgrande type	D/s Shell foundation Left bank	8 45
Total pressures	Electrical pressure cells, vibrating wire	Core	13*
	Pneumatic pressure cells	Core	2*
Total forces	Load cells, vibrating wire	Left bank only	8*
	Load cells, vibrating wire	Left bank only	3
	Load cells, strain gauge	Left bank only	4
Relative displacements	Extensometers Distofor type	Left bank only	7*
	Extensometers, multiple bars, micro metre reading (1 direction)	Left bank only	6
	Extensometers, single bar, electrical reading (1 direction)	Core	16
	Inclinometers (2 directions)	Core, u/s filter, d/s shell	5

Measured Parameters	Instrument Type	Instrument Location	Qty.
	Pendulums, inverted (2 directions)	Left bank only	2
	Joint displacement devices (3 directions)	Left bank only	60
Settlements (internal)	Magnetic discs around inclinometer tubes	Core (3 inclinom.) D/s shell (1 inclinom.)	130
	Pneumatic settlement cells	Core U/s shell	11
Displacements (by geodetic survey)	Bench marks (theodolite or distancemeter readings, levelling)	Crest, Berms on d/s shell Left bank	8 13 42
Accelerations (earthquake induced)	Strong motion accelerographs SMA-1, interconnected system	Crest, D/s shell slope Left Bank	1 2 3
	Peak acceleration recorders	Crest, D/s shell slope Left bank	7 4 1
	Seismic triggers, interconnected	Left bank	3
	Seismicscope	Crest, D/s shell slope Left bank	1 2 3
Seepage	V-notch weirs (remote reading planned)	Station at dam toe Left bank	2 6

Note: * indicates connection to the automatic data acquisition system

3.3 AUSTRALIA

Australia has approximately 409 large dams (*World Register of Dams*, 1988). The majority of the dams have been developed by state authorities responsible for the conservation and distribution of water for irrigation. These major authorities form the membership of the Australian National Committee on Large Dams (ANCOLD) (ICOLD, 1989). Overall, Australia has maintained a good dam safety record with the last dam failure having occurred in 1929. This safety record can be attributed to the fact that the majority of the dams are engineered, owned and operated by public authorities who have their own regular surveillance programs (Cantwell and Anderson, 1984).

In 1972, ANCOLD proposed that each state should legislate for a single control authority independent of the existing agencies which engineer and/or own dams. This control authority would insure that all dams are adequately designed, constructed, operated, maintained and monitored. By 1982 each state responded to some degree to ANCOLD's proposal. As of 1984 there was effective Dam Safety Legislation in only two states, New South Wales and Queensland. Furthermore, New South Wales is the only state that legislated for the aforementioned separate Control Authority (Cantwell and Anderson, 1984). However, the positive outcome of ANCOLD's actions is that today all of the public authorities responsible for dams have developed their own surveillance programs which include specifications for dam monitoring. For example, the Hydro-Electric Commission of Tasmania has a Safety Dam Unit (SDU) which performs the inspections of the dam, and reviews deformation surveys, instrumentation readings and leakages and maintenance data (Fitzpatrick et al., 1982).

Over the years ANCOLD has been the driving force in establishing Australia's dam safety practice. It has promulgated surveillance of dams. In fact, ANCOLD has published the following sources which reflect the current practice used in the surveillance and monitoring systems for the major dams in Australia (ICOLD, 1989):

1. ANCOLD (1976) - *Guidelines Operation, Maintenance and Surveillance of Dams.*
2. ANCOLD (1983) - *Guidelines for Dam Instrumentation and Monitoring Systems.*

A very comprehensive summary of these documents can be found in the report by ANCOLD in ICOLD (1989).

The complexity and extent of the monitoring system used on a dam is influenced by the "hazard rating" of the dam which is based on the potential economic loss and loss of life as a result of a structural or mechanical failure. However, once the monitoring system is selected it is assessed for conformity with the current standards of design, construction, maintenance and operation of large dams. After doing so, if required, a more comprehensive surveillance/monitoring system is installed. The ANCOLD Guidelines consider routine inspection and review systems fundamental to monitoring/surveillance and essential complements to the instrumentation system. In the design phase of large dams, the trend is to use numerical analysis techniques to estimate

the anticipated stresses and deformations exerted by the dam. This provides the logic for the design of the instrument systems in monitoring the conformity of the dam with acceptable behaviour patterns.

The systems which are considered in planning and development monitoring requirements for new and existing dams as described include:

(1) visual inspection and reviews,

(2) seepage measurement and analysis,

(3) groundwater/seepage pressure measurement,

(4) surface displacement and strain measurements,

(5) internal displacement and strain measurement,

(6) stress and load measurement systems,

(7) hydrometeorlogical, and

(8) seismicity monitoring.

A typical surveillance team consists of a competent staff made up of professionals and sub-professionals. For example, the Thomson Dam in Melbourne, Victoria, is regularly monitored by Caretakers, Engineers, Geologists and Surveyors (Robins and Walsh, 1989). Overall, the geotechnical and structural equipment used in these measuring systems vary from simple instrumentations (groundwater observation wells, strain gauges, extensometer, joint meters) to sophisticated instrumentations (e.g., pressure measuring tips providing registers of pressure at discrete locations or seismographs with timing accuracy of 0.01 second for periods up to one month, allowing very accurate location of earthquakes). Seismic monitoring is applied only to large dams. This is carried out by using or partly integrating the regional seismographic network and strong-motion sensor system installed randomly in the dam and its foundation. Surface displacements are determined by precise survey methods. These methods are used to register absolute movement of the dam and its abutments. Precise Survey systems are generally based on a triangulation and/or trilateration network with a high degree of accuracy. Different methods and accuracies are used for embankment and concrete dams. The number and position of survey targets, and the permissible accuracy and tolerances are determined by the designer, but the

methods and equipment used are determined by survey personnel.

The ANCOLD report gives some examples of the types of instrumentation systems used to monitor four of Australia's major dams. Table 3.2a illustrates some typical instrumentations used in five concrete faced rockfill embankment dams on the Pieman River on the west coast of Tasmania (Knoop and Lack, 1985). The frequency of the readings varies from very frequently during the construction and first filling to less frequently when the structure has reached a stable condition and recurrence pattern of behaviours are established. For unusual events and in special circumstances such as after an earthquake, or rapid draw down or flood conditions in excess of the normals, increased frequencies of observations is warranted (ICOLD, 1989). An example of such a monitoring program is given by Table 3.2b (Murley, 1983).

Table 3.2a
Number/Types of Instruments Installed in Five Concrete Face Rockfill Embankment dams on the Pieman River, Tasmania (after Knoop and Lack, [1985, p. 1107])

Type of Instruments	Mackintosh	Tullagardine	Murchison	Bastyan	Lower Pieman
Hydrostatic settlement cell	8	5	4	4	5
Survey targets	20	12	6	11	9
Crest clinometer base	3				
Face slope inclinometer	1		1	1	1
Perimetric joint meter	*4.12	2.5	8.16	3.6	3.8
Face joint meter, pin set	+12.34		8	4	10.10
Face strain meter	*20.34		5.29		
Embankment dilation meter	4				
Embankment pressure meter				*2.5	3.6
Foundation Piezometer	14				8
Leakage weir	1	1	1	4	6

Notes:
1. *4.12 means four location, twelve instruments installed in sets at orthogonal direction or in rosettes.
2. +12.34 means twelve joint meters, thirty-four sets.

Table 3.2b
Instrumentation Systems and Monitoring Frequencies[a]
(Based on Table 9, ANCOLD Guidelines for dam Instrumentation and Monitoring Systems)
(from Murley, [1983, p. 13])

Instrumentation and Monitoring System	During First Filling	Routine Monitoring Operational Phase	Remarks
Visual Inspection: check for cracks, settlement, slips	Daily	Daily and Weekly	Visual inspections by reservoir resident staff to be complemented by routine annual[d] inspection by surveillance and operation.
Seepage Measurement	Daily	Weekly (where risk/hazard rating of dam allows less frequent visual inspection, carry out seepage measurements in conjunction with visual inspections)	A complete series of readings of instrumentation monitoring systems should be taken at time of routine inspection.
Chemical Analysis of Seepage	If seepage is significant	6/month to establish seasonal patter of storage and seepage chemistry.	
Pore Pressure Measurement: foundations and dams.	Frequency as may be necessary to adequately define trends in behaviour with application water loading and development of seepage patterns.	3/month[b] Annually[c]	
Surface Displacement		6/month until seasonal pattern is established then annually.	
Internal Displacement		3/month[b] Annually[c]	
Internal Stress Measurement		3/month[b] Annually[c]	
Hydrometeorlogical	Operational requirement	Operational requirement	Continuity of monitoring may extend to service phase for major gated spillway operation.

Instrumentation and Monitoring System	During First Filling	Routine Monitoring Operational Phase	Remarks
Seismological	Continuous	Continuous	

Notes:

(a) Frequencies are generalised, particular circumstances, adverse trend in behaviour or risk/hazard ratings may dictate more frequent monitoring. After unusual event, such as rapid drawdown, filling or earthquake, carry out partial or full monitoring as appropriate.
(b) Suggested maximum interval, initial 3-4 years until dam and foundations exhibit stable characteristics.
(c) Maximum interval, subsequent years, regardless of satisfactory behaviour.
(d) After initial 5 years, subject to satisfactory behaviour, surveillance inspection interval may be increased progressively to 5 year interval.

In the latter part of 1980, the Department of Surveying and Land Information of the University of Melbourne conducted a study of Australia's automated management and improved presentation of dam monitoring data. This study concluded that a wide variety of approaches to automated management of data have been adopted by the majority of the state authorities and organizations responsible for the dam monitoring. The majority of these organizations have adopted PC systems as opposed to main frame systems. Overall, the automated recording of observations is considered well developed, but problems are still encountered with the automatic reading and recording of dam instrumentations (Sterling and Benwell, 1989).

Like most countries, the data from the numerous instruments within the monitoring system is processed and analyzed independently. The analysis often includes the recording of data in a continuous graphical form for ease of recognition of trends and comparison with design prediction of behaviour (ICOLD, 1989). The survey data from distance measuring instruments (e.g., EDM Wild DI-2000) and settlement measuring instruments (e.g., Jena Ni and 005A Ni-3 Automatic Levels) are also analyzed separately using developed or commercial software packages providing a least square solution (Sterling and Benwell, 1989). Case studies of existing dams by Knoop and Lack (1985), Barnet and Funnell (1983), and Fitzpatrick et al. (1982) illustrate some good examples of the types of graphical analysis used to compare the theoretical and measured data whereas Sterling and Benwell (1989) give examples of the software packages used for analysing the survey data.

3.4 AUSTRIA

Austria owns and operates approximately 123 dams, of which 80% (99) are concrete dams: 82 gravity and 17 arch (*World Register of Dams*, 1988). According to Duscha (1990), a survey by the ICOLD Committee on Dam Safety disclosed that Austria has indeed some form of dam safety legislation. In addition, the Austrian National Report (ICOLD, 1989) claims that in Austria the responsibility of dam surveillance is designated to an authority (e.g., General Water Right Authority) and that dam owners are obligated to assign a dam operator.

Furthermore, according to the Austrian National Report, dam owners have an obligation to inform the authority in charge of the dam surveillance of the organization selected to operate the dam. The designated operators are not only responsible for the overall operation of the dam and its pertinent structures, but also for the entire monitoring program. This includes recording and evaluating the measurements, preparing the annual or monthly reports, and maintaining the monitoring system. Consequently, for all of the dams within its jurisdiction, the surveillance authority has to review the reports and inspect each dam every 5 years (ICOLD, 1989; Ludescher, 1985). Based on the information provided by Duscha (1990) and the Austrian National Report in ICOLD (1989) it will be assumed that dam monitoring is in fact included in the Austrian regulations.

The majority information gathered on Austria reveals that Austrian dams are generally monitored using geodetic, geotechnical, and structural instrumentation. High precision geodetic surveys include levelling (of dam crests, slopes, embankments and abutments), alignment surveys (for non-curved gravity dams) and traverse (preferred for arch dams). Table 3.3 illustrates an example of one of Austria's more comprehensive surveillance system. The system is used to monitor Austria's largest dam, the Kölnbrein Dam (200 m high). Regrettably the percentage of the total number of dams that are currently monitored could not be determined from the information available. However, due to the fact that a large number of Austrian dams are situated in remote locations, more emphasize is placed on the installation of instruments that are

readily automated (i.e., geotechnical and structural instruments). A sample list of dams which have been automated is given in ICOLD (1989).

Table 3.3
Instrumentation of Kölnbrein Arch Dam in Austria
(from Ludescher, [1985, p. 799])

Quantities Measured	Instruments	No. of Instruments	Number of Reading Points	
			Total	No. Automated
Loads	Pressure balance for measuring reservoir water level	1	1	1
	Uplift pressure cells	41	41	25
	Stand pipe piezometer	154	154	124
	Concrete temperature transmitter	79	79	63
Displacement	Plumb line	17	34	17
	Clinometer	52	52	0
	Invar-wire extensometer	16	16	16
	Rod-type extensometer	137	137	76
	Sliding micrometer	26	982	0
	Contraction joint opening transmitter	115	137	0
	Geodetic points * levelling * traverse * target	205	262	0
Strain and stress	Teleformeter	84	84	64
	Telepressmeter	29	29	28
Flow	Leakage	12	12	12
Seismic Activity	Microseismic	1	1	1
	Microseismic	2	6	6
	Acoustic emission	2	4	4
	Meteorological data	7	7	7

Quantities Measured	Instruments	No. of Instruments	Number of Reading Points	
			Total	No. Automated
Total		980	2,038	444

According to the Austrian National Report (ICOLD, 1989), when considering any dam-type the following factors apply: (1) seepage and water pressure measurements constitute the main parts of the monitoring system, and (2) periodic geodetic measurements must be performed on all dams constructed in areas where there are potential landslides. With respect to the frequency of the measurements, the Austrian follow the guidelines suggested in ICOLD Bulletin N° 41. As an example, provided is Table 3.4 which gives the monitoring frequency program that has been applied to five large dams in the Glockner-Kaprun Hydro-Electric Power Development.

Table 3.4
Monitoring Frequencies for
the Glockner-Kaprun Hydro-Electric Power Development in Austria
(after Breitenstein et al, [1985, p. 1126])

Type of Measurement	Frequency
Inspection	Weekly
Control measurements	
Additional measurements	
Geodetic survey	Annually
Inspection with experts	
Inspection with authority	Every 5 years

Some of Austria's recent developments in the area of instrumentation for deformation monitoring include:

(1) acoustic emission devices to measure cracks in concrete or rock masses,

(2) laser plumb lines as an alternative to wire plumb lines,

(3) magnetic measuring devices to detect asphaltic concrete core wall deformations,

(4) special level indicating devices to determine the interior deformation of the dam, and

(5) an intelligent hand-held computer to record measurements of those instruments that have not been automated.

The above instruments have already been trialed and installed in new and existing dams. Another important piece of equipment that is included in most of the dam sites is a television camera. Considered as part of the automated system, the camera is used to provide an overall view of the dam structure (Breitenstein et al., 1985).

The analysis of the data consists of using deterministic models during the first filling, and a statistical-based mathematical model such as the multiple linear regression model after several filling periods (i.e. once more data has been collected). The results of the regression analyses are later used to improve the parameters of the deterministic model as well as to identify possible long term changes in the measured data. For the most part, the results of the analyses are presented in graphical and/or tabular form. The Austrian National Report in ICOLD (1989) suggest that the analysis of the data should be performed by dam experts and specialists with a statistical background.

For a more comprehensive review of the dam monitoring practices described in this section, the reader is referred to the Austrian National Report in ICOLD (1989). Furthermore, additional examples of monitoring systems used in Austrian dams are given by Schober and Lercher (1985).

3.5 BRAZIL

There are 516 large dams officially registered in Brazil. Seventy five percent of these dams (391) are embankment type dams (*World Register of Dams*, 1988). The most important dams are monitored using geodetic and/or geotechnical/structural instrumentation (Miguez de Mello, 1992).

There are no national or local standards/specifications for dam monitoring. The Brazilian National Committee on Large Dams has issued dam owners general instructions related to dam safety. These instructions are based on the guidelines published by the ICOLD (Miguez de Mello, 1992).

From a review of a number of articles by authors such as Seifart et al. (1985), Guedes and Coelho (1985), Caric et al. (1985), Lima et al. (1985), Filho et al. (1985) and Blinder et al. (1992) one can conclude that the instrumentation used to monitor earth and concrete dams are typical of those used by the majority of the ICOLD member countries. Overall, the monitoring schemes consists primarily of:

(1) geodetic benchmarks for high precision trilateration and levelling, and
(2) geotechnical/structural instrumentation including direct or inverted pendulum, thermometers, piezometers, electric strainmeters, electric jointmenters, electric stress meters, inclinometers, hydrostatic settlement cells and the most recent instrument the horizontal plate gauge used to monitor the horizontal displacement normal to the axis of the dam.

This review also indicated that dam monitoring in Brazil is predominantly based on geotechnical/structural techniques.

The analysis of Brazilian dams may consists of comparing the behaviour of the dam described by the data furnished by the monitoring instruments with that described either by FEM or by statistical model and/or a physical model (a miniature model of a selected section of a dam reproduced to scale). The data from the models is used as approximate limit values (Caric et al., 1985). The statistical model is based on the Gauss-Markoff Functional Probabilistic Model (H.G.M.). The H.G.M. model establishes a link between the effect variables (those characterizing the structural response) and the cause variables (e.g., upstream and downstream water level, ambient temperatures, concrete temperature, strain, stress, leakage flow). Some of the advantages of the H.G.M. model are that it is simple to use, it requires no knowledge of geometry of the structure nor the mechanical property of the materials and it can be used to model any kind of effect on any types of dams (Guedes and Coelho, 1985). Guedes and Coelho (1985) describe

the model in detail and provide several examples of how the model is applied to existing Brazilian dams.

Whichever model is used, the analysis is applied exclusively on points of interest within the dam. For example, the theoretical displacement at a point in the dam is compared to the actual displacement measured by a plumbline at that point. These results are commonly presented in a graphical or tabular form over time.

3.6 CANADA

Canada operates about 608 large dams, ranking amongst of the top ten in the world (*World Register of Dams*, 1988). The country has been blessed in that it has not yet experienced failures causing loss of life. However, there have been several incidents recorded which could have resulted in serious consequences had remedial measures not been taken (Koropatnick, 1990).

In Canada, the Canadian Federation empowers the ten provinces to develop and control their national resource. The management of water and power generation rests with the Provincial Government. The provinces are therefore responsible for dams in their jurisdiction, including licensing, regulation and public safety. In most provinces, dams are owned and operated by provincial "hydro" organizations. The Dam Safety Committee instituted in 1980 by Canadian National Committee on Large Dams (CANCOLD) undertook a study of regulations across Canada. The study revealed that all provinces and territories have enabling legislation that is very general in nature, lacking in specifics related to safety programmes and surveillance. Currently, Alberta, British Columbia (B.C.) and Québec are the only provinces that have developed specific safety practices supported by regulations and the administrative staff to implement them. (Dascal, 1991; Koropatnick, 1990; Tawil 1984; Brunner, 1983; *Dam and Canal Safety Regulations*, 1983). An excellent review of the laws and regulations governing the safety of dams in Canada is given by Tawil (1985) and Tawil (1984).

Alberta is the first province to institute laws governing safety of dams: the Dam and Canal Safety

Regulations enacted in 1978 (*Dam and Canal Safety Regulations*, 1983). The regulations are administered by the Dam Safety Branch which has produced guidelines for licensing and design flood criteria with respect to dam size and hazard potential. The legislation covers the inventory of approximately 1,200 dams more than eight meters in height or 60,000 cubic meters or more in capacity (Tawil, 1984; *Dam and Canal Safety Regulations*, 1983). Also, the Research Management Division of the Alberta Environment sponsored the Department of Surveying Engineering of University of Calgary to develop standards and specifications for deformation surveys (Teskey, 1988). A copy of the specifications is included in Appendix 8. These standards and specification relate only to geodetic monitoring surveys. They lack detail and technical information on the analysis, and are provided only as an example from which one can improve.

British Columbia used existing legislation to formulate a very comprehensive surveillance programm which includes the licensing, classification guidelines and inspection frequencies. In 1979, BC Hydro, the power authority in British Columbia, embarked on a study to evaluate 54 dams constructed before 1960. This evaluation lead to the remedial work of several dams between 30 and 60 years old. Also emerging from this appraisal, was the need to review B.C. Hydro's organization related to safety of dams. One of the most important outcome of the review is the creation of a new position of Director of Dam Safety in 1981 (Tawil, 1984). The Director's responsibility is to coordinate and oversee that all of the necessary steps are taken in order to fulfil B.C. Hydro's commitment to safety. Another aspect of the B.C. Hydro's Dam Safety Program is the preparation and updating of Operation and Maintenance Manuals for all of its dams. B.C. Hydro is said to be committed to maintain safety standards consistent with standards such as those established by ICOLD (Brunner, 1983).

In Québec, the Hydro-Québec dam safety polioy was enacted in October 1985. The act defines the principles regarding the operation, surveillance and maintenance of dams in order to prevent or limit the consequences of potential failures. The act is implemented by the Civil Works Division in each region and by the Dam Safety Directorate from the Generation, Transmission and Distribution Branch. Since the implementation of the act, two very significant objectives have been achieved. The first objective was achieved at the end of 1990 when the Dam Safety

Directorate had implemented nine technical regulations, sixteen standards and three procedures concerning dam surveillance and safety evaluation. The second objective achieved is that a list of terms and expressions used in dam safety has been drafted with the aim to provide precise definitions and allow standardization of dam surveillance and behaviour evaluation throughout the province. Another major activity of the Dam Safety Directorate is the training of surveillance personnel. The Directorate has provided courses to field inspectors up to the end of 1990 while training programs for the engineering staff is scheduled to start this year in 1992, if not already started (Dascal, 1991).

In the province of Manitoba, the Manitoba Hydro became particularly involved in dam safety in 1974 when surveys of two older dams (Seven Sisters and Great Falls) revealed extensive concrete deterioration which presented a serious safety hazard. As a result rehabilitation programs were started in 1978 and a formalized Safety Surveillance program was approved in 1979. Although the new program provided guidelines for monitoring the conditions and ongoing performance of various structures, it was not complete in providing an overall dam safety program. The program was eventually expanded in 1987 to include a number of safety guidelines published by the US Corps of Engineers (COE) and those listed in ICOLD's *Check List on Dam Safety*. One of the components that was adopted by Manitoba Hydro is that for each dam, the instructions on the instrumentation and monitoring is to be included in the dam Operating and Maintenance Manuals (Koropatnick, 1990). According to Koropatnick (1990), the current program is consistent with modern day practices in Dam Safety Engineering.

In the remaining provinces, progress towards dam safety varies greatly. The need for programs to ensure public safety is recognised but the overall progress is slow and usually hindered by inadequate funding. Although one may conclude that, in general, Canadian dams are adequately attended, some shortcomings still persist (Tawil, 1984).

Some of the common problems with Canada's dam safety programmes include poorly designed monitoring schemes, inadequate instrumentation, lack of calibration facilities, and insufficient accuracy of measurements. One of the most serious problems which requires immediate

attention, at the national level, is the use of out-dated methods in the geometrical analysis of deformation measurements (Chrzanowski, 1990). The physical interpretation of deformation surveys requires knowledge from a matrix of interdisciplinary experts.

There are two known institutions in Canada that have significantly contributed to the progress in the area of integrated monitoring and analysis of deformations. These are the Surveying Engineering Departments at the University of New Brunswick (UNB) and University of Calgary.

Over the past fifteen years, the Engineering and Mining Surveying Research Group at UNB has been intensely involved in interdisciplinary research in the development of new techniques and new methods for the integrated monitoring and analysis of deformations in engineering and geoscience projects. The research has led to the development of an earlier described (section 2.3.3) UNB Generalized Method for Deformation Analysis (Appendix 4). The method is supported by a software package DEFNAN written in FORTRAN 77 which can be executed either on an IBM 3090 mainframe or on an IBM compatible Personal Computer. As aforementioned, the UNB Generalized Method can be applied to any type of structures and it utilizes different types of geodetic and geotechnical measurements in a simultaneous analysis. This method has been successfully implemented in many types of engineering and geoscience projects. For example, it has been used for ground subsidence studies, in oil fields in Venezuela and in mining areas in Canada, Poland and China; and for deformation analyses of concrete dams and earthfill dams in the USA, Canada and Venezuela. In dam deformation studies in Canada, the UNB Generalized Method has been applied extensively in cooperation with NB Power in an integrated analysis of deformations of the structures at the Mactaquac hydro-electric power generating station in New Brunswick (Chrzanowski et al., 1989). The method, though used by NB Power, has not yet been adopted by other provinces. The main reason being generally the inadequate educational background of those in charge of the analyses.

The current research in UNB is focused on an optimal combination of deterministic modelling of deformation with results of the geometrical analysis of deformation observations for the integrated analysis and physical interpretation. The deterministic modelling is supported by

software FEMMA for 2-D and 3-D elastic and visco-elastic finite element analysis (Szostak-Chrzanowski and Chrzanowski, 1991). A similar method, developed by the University of Calgary is said to perform an integrated analysis of deformations by combining a physical model (finite element) of the structure with the actual deformation measurements on the structure. The method has been applied to large fill dams, the Calgary Olympic Oval and large diameter buried pipelines (Biacs and Teskey, 1989). The research at both UNB and University of Calgary has put Canada into a leading position in international developments of new methods for deformation analysis.

Although there are some shortfalls in the way Canadians maintain and monitor their dams, Canadian organizations such as CANCOLD and the Canadian Dam Safety Association (CDSA) are continuing their efforts to increase the awareness of dam safety through annual conferences and publications. The CDSA was originally instituted by CANCOLD to assemble and review information on the existing rules and regulations governing dams in various parts of Canada. One of its key findings was that although all jurisdictions have enabling legislation which set the responsibility for operation and maintenance with the dam owners, the legislation is not specific on monitoring and surveillance (Tawil, 1985). CDSA also organizes annual conferences where its members have an opportunity to present papers and exchange information on a number of subjects ranging from performance monitoring to legislation and remedial works (Anderson, 1990). CANCOLD contributes to dam safety in Canada through its membership with ICOLD and its participation in some of ICOLD's special committees. As members of ICOLD, CANCOLD is in an ideal position: it has direct access to ICOLD publications; it is in direct contact with an overwhelming number of international experts on dams; and it can exchange information with the other 78 or so ICOLD members in an unilateral, bilateral or multilateral agreement. One of CANCOLD's latest report is its national report to the ICOLD Committee on Monitoring of Dams and their Foundations published in ICOLD (1989). The report is CANCOLD's views on the instrumentation concepts and installations related to new and existing large Canadian dams. Unfortunately CANCOLD's report in ICOLD (1989) is very general in nature, however it does bring forth some concepts that Canadians believe to be an important part of a dam monitoring program. The report omits the aforementioned developments at the

University of New Brunswick and Calgary which work within the activities of FIG rather than ICOLD.

Overall, Canada is in agreement with most countries in that the extent and scope of instrumentation to be installed in a dam should be determined early in the design stage. Canada also recognizes that knowledgeable people should be assigned to the surveillance program to ensure that the monitoring of the dam and reservoir slopes is properly carried out. The types of monitoring instruments that should be included in concrete and embankment dams, as recommended by CANCOLD, are summarized at Table 3.5. With respect to the monitoring frequencies CANCOLD recommends the following: during filling piezometers and seepage weirs are to be read once-a-day, others once-a-week and surface surveys once-a-month. These frequencies are to be maintained until the reservoir has reached full pool and for a few months after. Once the structure has reached a normal pattern of behaviour, the frequency of the readings may be lengthened to suit the requirements of the design engineer (ICOLD, 1989). Examples of monitoring frequencies that have been implemented on two Canadian dams are given at Tables 3.6a (Revelstoke Dam), 3.6b (La Grande Complex) and 3.6c (Bennet Dam). Table 3.6c also includes the obtained accuracy of the measurements. CANCOLD comments on some of the advantages and the requirements of automated monitoring systems, however it does not express whether or not these systems have been implemented in Canadian dams. CANCOLD's report falls very short in what was stated to be one of the key elements to a successful monitoring program, the analysis of the observation data. The report simply states that: during the filling of the reservoir the data should be analyzed as soon as it is available; the observers should be trained to take accurate observations and not to cover-up any reading that appear to be inconsistent with the previous readings; and the data should be documented and given to engineering office for assessment (ICOLD, 1989).

For more information on the types of monitoring systems applied to some of Canada's dams, the reader is referred to a number of articles written by CANCOLD members in the ICOLD Proceedings of the Fifteenth Congress on Large Dams, Q56, Vol I (e.g., Klohn et al. 1985; Eisenstein and Brandt 1985; Taylor et al. 1985), in the Proceedings of the 5th International (FIG)

Symposium on Deformation Surveys (Chrzanowski et al., 1988; Wroblewicz et al., 1988), as well as in the Proceedings of the International Conference on Safety of Dams (Paré, 1985).

Table 3.5
Instruments that Should be Used to Monitor
Concrete and Embankment Dams: As Recommended by CANCOLD
(after ICOLD, [1989, pp. 77-80])

QUANTITIES MONITORED	INSTRUMENT	REMARKS
CONCRETE DAMS		
Deformation	Surface survey points	* located on the dam crest and along the d/s face to monitor both horizontal and vertical movement
	Plumblines, pendulums, plummets	* located in wells on shafts to monitor tilt
	Rod-extensometers	* extending from the base of the dam into the rock to monitor foundation displacement
	Inclinometers	* casings drilled into bedrock and grouted into the dam to monitor foundation movement
	Tiltmeters/plumblines	* located in concrete mass to measure tilt movement (a series of electric tiltmeters can be used to replace a manually read plumbline)
	Jointmeters	* imbedded into the concrete to monitor opening or closing of contraction joints and cracks
Temperature and Stress/Strain Measurements	Thermometers	* should be embedded to measure mass concrete temperature changes
	Construction Thermometers	* located throughout the dam to monitor temperature during the construction (they are abandoned as construction proceeds)
	Stress and Strain Meters (Carlson strain meters, vibrating wire strain gauges, resistance type gauges)	* a minimum of 3 stress meters are required to calculate the principal stresses (vertical, horizontal and 45°) * they are not to be used in concrete dams with height below 25 m and anticipated stresses below 700 Kpa * strain meters may be used in areas that may be in tension
Piezometric Pressure and Water Flow Measurement	Piezometers and other appropriate instruments	* devices to measure piezometric pressure in shear zones in foundation rock, at concrete/rock interface and adjacent to penstock

QUANTITIES MONITORED	INSTRUMENT	REMARKS
	Electronic open-trench-type flumes	* to measure flows in the galleries from the foundation drains and leaks in the dam
EMBANKMENT DAMS		
Pore Pressure	An appropriate type of piezometer to suit the location and material in which the pressure is being used	* should be accurately measured at locations where seepage occurs * in some cases it is best to isolate dam seepage into various areas of the dam by constructing isolation dykes within the dam
Deformation Measurements	Surface settlement points	* in general as per concrete dams
	Vertical measurement gauges, extensometers	* usually anchored to bedrock and rise as the fill is raised
	Slope indicator devices	* to measure horizontal movement transverse and parallel to the axis
	Hydraulic settlement devices	* installed within the dam fill provide an alternate means of measuring the internal consolidation of the fill or foundation
Horizontal Deformation Within the Fill	Aquaducer probe, Cross-arms (another means is by operating a latchcone inside a horizontal movement gauge casing installed perpendicular to the dam axis in the d/s shell)	* measured in critical areas where the dam fill is subject to large horizontal loadings
Stress Within Dam Fill		* placed at certain critical locations like steep abutments or narrow gorges and at earthfill-concrete interface
	Earth pressure cells	* earth pressure cell measure the total pressure
	Earth pressure cells installed with piezometers near the cell	* earth pressure cell with a piezometer installed near it gives the effective stress * these measurements give indications if hydraulic fracture of the fill or arching between internal zones is taking place
Horizontal Strain Measurements	Horizontal strain gauges	* should be placed in areas of potential tensile cracking such as steep abutments and abrupt changes in elevations of foundation rock * they are usually anchored at one end of the dam in the abutment rock or to concrete abutment structures

Note: Other quantities that should be measured include reservoir level, ambient temperatures, rainfall and snowfall

measurements, frost depth penetration and seismic activities (a must for all large dams).

Table 3.6a
Frequency of Instrument Readings
at Revolstoke Dam (Canada)
(from ICOLD, [1989, p. 86])

Stage		Instrument				
		Core piezometer	Foundation and shell piezometer	Vert. movement gauge	Hor. Movement gauge	Hor. Strain gauge
During Construction		frequently by field staff				1/month
Reservoir filling		1/2 days	1/2 days	1/week	2/month	2/month
After the first res. filling	first 6 months	1/week	1/week	1/month	1/month	1/month
	6 months to 1.5 yrs	2/month	2/month	4/year	4/year	4/year
	1.5 to 2.5 years	2/month	2/month	4/year	4/year	4/year
	2.5 to 6.5 years	6/year	6/year	2/year	2/year	2/year
	subs. years	2/year	2/year	1/year	2/year	1/year

Stage	Instrument				
	Surface Monument	Earth Pressure Cell	Weir and Well	Strong Motion Accelerog raph	Visual Inspection
During Construction	1/month	1/month	1/month	continuous	1/month
Reservoir filling	1/month	2/month	1/2 days	continuous	1/day

Stage		Instrument				
		Surface Monument	Earth Pressure Cell	Weir and Well	Strong Motion Accelerograph	Visual Inspection
After the first res. filling	first 6 months	1/month	1/month	1/week	continuous	1/day
	6 months to 1.5 yrs	4/year	4/year	2/month	continuous	1/week
	1.5 to 2.5 years	4/year	4/year	2/month	continuous	1/week
	2.5 to 6.5 years	2/year	2/year	6/year	continuous	2/month
	subs. years	1/year	1/year	2/year	continuous	1/month

Table 3.6b
Frequency of Instrument Readings
at La Grande Complex (Canada)
(from ICOLD, [1989, p. 88])

STAGE		INSTRUMENTS						
		Sealed Piezometer	Casagrande Type Piezometer	Inclinometer	Frost	Surface Monument	Extens., Total Pressure Cell and Settlement Cell	Weir
During Constr	Constr. Period	1/month	1/month	6/year	-	-	1/month	-
	Interim Period	1/season	1/season	1/season	-	-	1/season	-
Res. Filling		1/2 days	1/day	2/month	1/ month	2/month	2/month	2/ week
After the 1st Res. Filling	First Year	1/week	1/week	6/years	1/ month	1/month	1/season	2/ week
	Second Year	2/month	2/month	1/season	1/ month	1/season	1/season	1/ week
	Third Year	6/years	1/month	2/year	1/ month	2/year	2/year	1/ month

STAGE		INSTRUMENTS						
		Sealed Piezometer	Casagra-nde Type Piezometer	Inclinometer	Frost	Surface Monument	Extens., Total Pressure Cell and Settlement Cell	Weir
	Subs. Year	2/year	4/year	1/year	1/ month	1/year	1/year	6/year

Table 3.6c
Instrumentation Frequency and Accuracy
at Bennet (Embankment) Dam in British Columbia (Canada)
(from Taylor et al., [1985, p. 189])

INSTRUMENT	NO. INSTALLED	ACCURACY	FREQ. OF MONITORING
Foundation piezometers (standpipe 26, pneumatic 4)	30	± 3 mm	quarterly
Embankment piezometers (telmac 8, hydraulic 38)	46	± 3 mm	quarterly
Stress/strain meters	4/11	1 unit	quarterly
Cross-arm Devices	2	V - ± 3 mm	annually till 1975
Slope indicators	10	H - ± 1 mm	less frequently since 1975
Surface settlement pts	54		
Levels		± 12 mm √(km)	Annually
Offsets		± 15 mm	Annually
Displacement points	17	± 3 mm	Annually
Survey control		± 4 mm √(km)	Annually
Seepage measurements	•		
Flumes	7	± 3 mm	quarterly
Weirs	3	± 3 mm	weekly

3.7 CHINA

China is an ancient country with a long history of dam construction dating back as early as 240 B.C. (Junchun, 1985). In 1988 China registered 18,820 large dams, about 52% of the total dams within the 79 ICOLD member countries (*World Register of Dams*, 1988). The status of monitoring these dams can be summarized as follows (Chonggang, 1992):

(1) 90% of the reservoirs with a capacity larger than 100 million cubic meters are monitored,

(2) 30-40% of the reservoirs with a capacity between 10-100 million cubic meters are monitored, and

(3) reservoirs with a capacity below 10 million cubic meters are not monitored at all.

China's latest contribution to the construction of dams has been the development of the roller compacted concrete (RCC) method. This method of construction, initiated in 1979, has been proven to be very economical. For instance, the construction period of the Kengkou Dam, the first RCC gravity dam completed in 1986, was reduced by about one year, and the investment cost by about 17%. Today there are at least eight RCC dams in operation with the first RCC arched gravity dam scheduled to be completed in 1992. The instrumentation systems for RCC dams are common to standard concrete dams. Despite its economical advantages, the RCC method requires further research regarding the analysis of the surveillance data, the possibility of constructing very high dams (greater than 150 meters) and the problems associated with the behaviour and mixture of the concrete (Chonggang, 1991).

China has placed deformation surveys high on its priority list. In the past it has experienced tragic events resulting from unpredicted failures of engineering and geological structures. Consequently, the State has implemented specifications for monitoring engineering structures, mining facilities and crustal movements. Currently China has national regulations directing that any large engineering structure must be monitored (Chen, 1988). The specifications and regulations that govern the monitoring of Chinese dams are (Chonggang, 1992)

1. *Regulations of Reservoir Safety* (national standards published in Chinese in 1991).
2. *Technical Specifications for Monitoring of Concrete Dams* (published in 1992 in Chinese).
3. *Specifications for Embankment Monitoring* (still being written).

4. *Interim Statute for Dam Safety Management of Hydropower Stations* (edited by Large Dam Safety Supervision Center, Ministry of Energy in Hangzhou).

The accuracy, procedures and survey frequency for different types of structures are in accordance to the specifications issued by the corresponding State ministries. Table 3.7 is a summary of the specifications used for monitoring dams (Chen, 1988).

In China, each power station employs a survey team of five to ten persons to routinely monitor the deformation of the dam. The majority of the engineers responsible for the supervision of the deformation surveys have a bachelor degree in surveying or geodesy (Chen, 1988).

The Chinese plan the monitoring scheme during the design stage of the project. Conventional geodetic surveys using EDM Instruments, with ranges up to 50 km and accuracies of 5 mm ± 1 ppm, are widely used. Geotechnical/structural monitoring devices include traditional instruments (e.g., strain meters, stress meters, pore water pressure gauges and thermometers) and/or recently developed instruments (e.g., telemetric coordinameters capable of measuring displacements up to 50 mm with an accuracy of ± 0.10-0.18 mm, remotely controlled laser alignment systems and fully automatic high-precision hydrostatic level tiltmeter capable of measuring tilts with an accuracy of 0.001 second of arc) (Chen, 1988). In addition to using instruments and surveillance methods stipulated by the specifications, considerations are given to design and implementation of specific monitoring schemes according to unique features of the structure or its foundation (Dehou and Quanlin, 1985). Dehou and Quanlin (1985) present a good example of the monitoring scheme and the data analysis used for the Gezhouba project in the Three Gorges of the Yangtze River.

Table 3.7
Main Requirements for Dam Deformation Surveys
in China
(after Chen, [1988, p. 140])

	Concrete Dams	Earth-rockfill Dams
Quantities Monitored	* Foundation subsidence & tilt deflections * Horizontal displacements * Pore water pressure * Seepage * Temperature of concrete * Stresses of the concrete	* Horizontal displacements * Vertical displacements * Pore water pressure * Seepage
Monitoring Accuracy	Horizontal displacement: 1.0 - 1.5 mm Vertical displacement: 1.0 - 1.5 mm	* During construction Horizontal displacement: 10.0 mm Vertical displacement: 5.0 - 10.0 mm
		* During Operation Horizontal displacement: 5.0 mm Vertical displacement: 3.0 - 5.0 mm
Monitoring Frequency	During Filling of the Reservoir	
	7.0 - 10.0 days	7.0 - 10.0 days
	From Full Filling to Achieving Stability (3 - 5 yrs)	
	0.5 -1.0 month	1.0 month
	During Normal Operations	
	1.0 - 3.0 month	3.0 months

China's experience in automated monitoring systems is limited to automatic data collectors. However, research experts such as Dehou and Quanlin (1985) believe that establishing an automated monitoring system capable of interpreting structural behaviours correctly and timely is extremely important for the safety of large hydraulic structures.

In China, the common practice is to integrate different survey techniques to detect, locate and eliminate any gross errors introduced by surveys. For instance, on top of a concrete dam, the horizontal and vertical displacements of a construction block can be determined using optical

alignment and precise levelling. The displacements from this survey can then be compared with the displacement in the galleries determined, for example, by laser and tensioned wire alignment systems (Chen, 1988).

The physical interpretation of dam deformation surveys is realized by statistical models, deterministical models using the Finite Element Method (FEM) or the combination of both. Due to the uncertainties of deformation and ignorance of non-elastic behaviour in the deterministic model, the calculated displacements will generally depart from the observed values. Thus, the deterministic model is improved using a statistical method (least square solution) that estimates the residuals and the unknown coefficients of the model (Chen, 1988). With the current methods of data analysis, the observations from geodetic, geotechnical and structural systems are processed and analyzed separately. The geodetic data is processed and analyzed by surveyors and the goetechnical and structural data by civil engineers. The reader is referred to Junchun (1985) for some practical examples of monitoring and analysis of earth and rockfill dams in China.

3.8 FRANCE

France has 468 large dams officially registered. Over half the 468 dams (310) are concrete dams comprising primarily of gravity dams (188) (*World Register of Dams*, 1988). There are numerous reports and case studies written on the safety and monitoring of French dams. A large number of these reports have been published in the Proceedings of the ICOLD Congresses but, consequently the majority of them, if not all, are written in French without English versions. The only English written report found on the status of dam monitoring in France is the one submitted by the French National Committee on Large Dams to the ICOLD Committee on Monitoring of Dams and their Foundations. This report was published in ICOLD (1989) and provides a fairly comprehensive overview of dam monitoring in France. The report is primarily based on the experience in dam monitoring of the Electricité de Françe (EDF) who owns and operates over 150 large dams in France. A summary of the French National Committee's report is provided in the proceeding discussion.

France is an example where government intervention has resulted in the development of an effective and homogeneous system of inspection and surveillance of dams. The French law, dated 14 August 1970, requires inspection and surveillance of all dams more than 20 metres high and those whose failure would result in the loss of human life (ICOLD, 1989). The French Legislation on Inspection and Surveillance of Dams has been reproduced and included as Appendix 9. The components of the law which are believed to be fundamental to dam monitoring are (ICOLD, 1989):

(1) the surveillance of the dam is the responsibility of its owner,

(2) a special Government Department ensures that the surveillance is properly carried out by the dam owner,

(3) the Government Department makes its own visual inspections each year, and a full inspection five years after first filling and every ten years subsequent, and

(4) for older dams the Government Department has a prioritised list of the dams to be reviewed with which it invites the dam owners to submit a report to the Permanent Technical Committee on Dams containing the recommendations on the instruments to be installed.

The French power agencies are self contained and incorporate the following typical divisions (ICOLD, 1989):

(1) a Construction Division which undertakes the design and construction of the dam, and it is given the responsibility for dam safety during the construction and first filling,

(2) a Generation and Transmission Division who is in charge of the operation and maintenance of the dam, and

(3) a Research and Development Division who is responsible to conduct studies and tests on materials and structures, and it ensures that it is informed on the results of monitoring.

Some of the additional responsibilities of the divisions are the installation of instruments, collecting the measurements, data processing and interpretation of the results.

In order to ensure that close liaison is maintained between all of the departments there is a Monitoring Committee consisting of a representative from all of the divisions. The Committee's

official role is to coordinate monitoring policy and practices between the divisions, and to disseminate information on existing and proposed installation. Note that this does not exclude the fact that if required outside experts may be called upon.

Some of the key principles of dam monitoring in France include simplicity, reliability (of instruments and measurements), redundancy, selective monitoring (i.e., read a selective number of key instruments) and semi-automation (i.e., interpretation of measurements and safety evaluation remains to be a task of an engineer). The monitoring schemes adopted by the French comprise of both geodetic, and geotechnical and structural instrumentation. However, depending on the type of dam, one method is preferred over the other. For example, for arch dams the policy for monitoring is to use primarily geotechnical instruments. The argument against the use of geodetic methods is that in France most arch dams are situated in difficult terrain or in high mountainous areas where it is extremely demanding for skilled personnel to access the sites with high precision instruments. In addition, with geodetic methods there is a necessity to check the observation stations against other stable reference points. On the other hand, for arch dams, surveys are used only on those dams with thin arches where pendulums cannot be incorporated in the structure (e.g., in the shafts and galleries of the dam) (ICOLD, 1989).

For concrete and embankment dams the instrumentation used by the French to monitor the dams consist mainly of:

1. **Concrete Dams.** Surface monuments for levelling and trilateration (using EDM), pendulums (direct or inverted plumblines); extensometers; strain meters (particularly in multiple arches and in buttresses); inclinometers; piezometers; pore pressure cells (in soils); and seepage measurement devices.
2. **Embankment Dams.** Survey targets fixed on concrete beams sunk deep into the fill for plane triangulation and direct levelling; internal movement measuring devices such as hydraulic level, extensometers with vibrating wire or induction sensors, and electromagnetic torpedoes in horizontal tubes; and open pipe piezometers, and hydraulic (USBR type) or electric pore pressure cells.

Examples of existing dams that are being monitored using these types of instruments can be

found in ICOLD (1989).

The most recent advancements have been developments in telemetric data acquisition which include telependulum (induction type), distofor and extensofor. The telependulum is basically a pendulum that can be read remotely. The distofor yields a near continuous log of movements along a borehole with no friction between the sensor and the bore hole. The extensofor is similar to the distofor except that is mobile and is lowered into the hole each time a reading is taken (ICOLD, 1989). A more recent method of monitoring which has been experimented on the Riou Dam in the French Alps, France's second roller compacted concrete (RCC) dam, is thermodynamic monitoring. This method consists of placing in groups every five meters vertically, thermometers, extensometers and total pressure cells in selected cross-sections of the dam. In the Riou Dam, the analysis of the results obtained showed the effect of external temperature during construction and the disturbances caused by the phenomena at the dam face (Goubet and Guérinet, 1992).

During the construction and filling the initial analysis of the data consists of plotting the observations of significant parameters against time, concrete or fill level and reservoir level (e.g., concrete strain against reservoir level). Subsequently as more measurements are collected more detailed statistical analysis are done. For example, for concrete dams one can determine the reversible elastic effects of concrete due to the changes in water level and temperature. Also, using the relationships between the displacement temperature relationship and the displacement water level relationships one can correct time related movements for water level and temperature effects. These time related movements are then plotted on a graph and the behaviour of the dam is considered to be normal if and only if the movements fall within a specified dispersion band.

Another approach that the French have taken in the analysis of data is to compare the instrument readings with those calculated from FEM analysis during the design. For fill dams a back-analysis approach is applied where the in situ measurements taken during and/or after construction are used to calibrate the deterministic model and make it react in the same way as the dam. The back-analysis approach is considered to be better for predicting the behaviour for

future stages of construction and operation.

After a number of years, with a large quantity of data available, statistical analyses modelling (see section 2.3.4) is done. Eventually *performance models* are developed by fitting an equation to the results using hydrostatic load, time of measurement, and time measured from the original measurement as variables. These models are then used to separate reversible temperature and load effects from the irreversible. Examples of this type of analysis can be found in ICOLD (1989).

3.9 GERMANY

Germany has a total of 261 large dams officially listed in the *World Register of Dams* (1988). The only information found on the dam safety program in Germany is that of the state of Nordrhein-Westfalen. The material presented hereinafter is based on a number of technical papers written by members of the German International Committee on Large Dams and other German experts in the field of deformation monitoring.

In January 1990, Germany hosted a conference to address several issues related to the safety of dams in the state of Nordrhein-Westfalen. In Nordrhein-Westfalen the water authorities are responsible for inspecting and maintaining the safety standards of the dams. The state regulations stipulate that a dam has to meet "generally recognized technical standards" based on the current technology, as well as on the experience of specialists in the field. These standards have been established and published by a number of scientific and technical organizations, in particular the Deutsche Institut für Normung (DIN). Recently, however, it has been recognized that the key areas requiring additional attention are: (1) a review of monitoring and control measurements, and (2) the need for establishing guidelines for the frequency of measurements and methods for evaluating the results. Guidelines prepared by a working group of experts from the State Dam Committee have already established the basis for setting the minimum level for monitoring and control (IWP & DC, 1990).

Articles written by Idel and Wittke (1991), Rissler (1991), and Schenk (1988) clearly illustrate that German dams are monitored using classical geodetic, and geotechnical and structural techniques. However, the inclusion of geotechnical instrumentation to monitor dams has been a recent trend that has only begun in the past ten to fifteen years. For example, the Fürwigge Dam was not monitored with geodetic equipment until 1986 as part of the German Dam Safety Program (Rissler, 1991). Idel (1982) shows that primitive methods such as simple open and/or pumped drains were still being used to determine seepage and hydrostatic pressures for earthfill and masonry dams up to the latter 1970. Furthermore, although modern dams such as the Primstal Dam (1981) have been designed with both geodetic and geotechnical instrumentations, the German geotechnical and structural monitoring systems when compared with those used on Italian dams, seem to be inferior. Table 3.8 illustrates the detailed monitoring program applied to Primstal Dam.

The Germans, like the Swiss, still seem to place more emphasize on geodetic measurements. For example, with respect to new advancements in the area of monitoring techniques, the emphasis has been on the development of new geodetic systems. Some of the modern techniques proposed include automatic theodolite/tacheometer systems, terrestrial photogrammetry and differential GPS (Niemeier and Wunderlich, 1988). Niemeier and Wunderlich (1988) go as far as suggesting a future monitoring system comprising of the combination of differential GPS-positioning and automatic theodolite/tacheometer system. The authors claim that this system will be able to determine the actual position of the dam in real time and that it will also be possible to connect the system with internal geotechnical monitoring devices (e.g., centring the GPS antennas above the plumbline shafts). Table 3.9 shows the comparison of the three proposed methods as well as the classical triangulation/trilateration techniques.

Table 3.8
Monitoring System of the Primstal Dam (Germany)
(from Schenk, [1988, p. 765])

Measurements	Instruments and Location	Intervals of Measurements	Remarks
Deformation of Dam and Dam Axis	Settlement gauges, Displacement gauges, horizontal geodetic measurements at sealing face	2/a (2 per year) 1/a (1 per year)	Until guarantee time (1987) After 1987 and during impoundment at every 10 meters drawdown
Settlements of Concrete Structures	Geodetic measurements of gallery and bottom outlet	1/a (1 per year)	
Stress Measurements	Total pressure gauges, bottom outlet	1/a (1 per year)	
Discharge of Drainage System	Drains of asphaltic sealing, Concrete drains, drainage shafts d/s	daily weekly	Visual control Measurement
Inspection	Plant	daily	
Hydrochemical Analysis	reservoir and drainage system, outside piezometer	2/a (2 per year)	
Piezometer, Observation Wells, Natural Springs	Gallery, abutments and d/s slope, valley floor, discharge of piezometer	daily weekly	Visual control Measurement

Table 3.9
Critical Comparison of Various Geodetic Techniques (German)
(from Niemeier and Wunderlich, [1986, p. 385])

Approach	**Terrestrial (Geodetic)**		**Terrestrial Photogrammetric**	**Satellite Supported**
Method	**Triangulation Trilateration**	**Automatic Monitoring**	**Close Range Photogrammetry**	**Differential GPS Phase Measurements**
Overall Accuracy	$\leq \pm$ 1-2 mm	$\leq \pm$ 3 mm	$\leq \pm$ 6 mm	$\leq \pm$ 3 mm

<table>
<tr><th>Approach</th><th colspan="2">Terrestrial (Geodetic)</th><th>Terrestrial Photogrammetric</th><th>Satellite Supported</th></tr>
<tr><th>Method</th><th>Triangulation Trilateration</th><th>Automatic Monitoring</th><th>Close Range Photogrammetry</th><th>Differential GPS Phase Measurements</th></tr>
<tr><td>Reliability and Sensitivity</td><td>Familiar and approved procedures to ensure success. Clear terms and measures of assessment.</td><td>Less control stations. More frequent observations. Prediction by filters. Alternatives at total system breakdown?</td><td>Not sufficient for targets. Non-targeted points determinable. Subsequently calibration parameters are part of solution.</td><td>4 observation limit. Redundancy demands at least 3 receivers and sophisticated planning of session sequence. Transformation problems for subsequent terrestrial surveys.</td></tr>
<tr><td>Cost</td><td>Skilled survey parties. Precision instruments can be dedicated to various other purposes.</td><td>High expenditure. Attractive rate of utilization. Immobility drawback.</td><td>Customary camera. Precision comparator.</td><td>Still exclusive purchase price of receivers. Receiver pools? Experienced operator for data processing.</td></tr>
<tr><td rowspan="2">Usefulness</td><td colspan="2">Only targeted points suitable for measurement.</td><td rowspan="2">Conclusive survey: behaviour of entire structure.</td><td rowspan="2">Accessible sites free from obstructions. No electronic disturbances.</td></tr>
<tr><td>Common deformation analysis programmes.</td><td>Simultaneous monitoring of sliding slopes</td></tr>
<tr><td rowspan="2">Connection</td><td>Conventional</td><td>Conventional</td><td>Conventional</td><td>Antenna above plumbing shaft.</td></tr>
<tr><td>Strong link if loop-holes in top gallery</td><td colspan="3">Difficult or poor link between survey systems inside and outside the dam.</td></tr>
<tr><td>Long-term Stability</td><td>High effort to keep EDM-correlation under control.</td><td>Changes will be quickly revealed.</td><td colspan="2">Several reference distances (loss of reference mark involves transformation problems)</td></tr>
<tr><td>Absolute Stability</td><td colspan="3">High additional effort, often not practicable.</td><td>No problem if dual ban receiver.</td></tr>
<tr><td>Repetition</td><td>Seasonal</td><td>Seasonal</td><td>Seasonal</td><td>Probably continuous.</td></tr>
<tr><td rowspan="2">Observation Time</td><td>3 days</td><td>In minutes</td><td>1/2 day</td><td rowspan="2">Depends only on system availability and number of receivers.</td></tr>
<tr><td colspan="3">Heavily dependent on weather conditions.</td></tr>
</table>

Approach	**Terrestrial (Geodetic)**		**Terrestrial Photogrammetric**	**Satellite Supported**
Method	**Triangulation Trilateration**	**Automatic Monitoring**	**Close Range Photogrammetry**	**Differential GPS Phase Measurements**
Processing (response time)	Zero epoch: in days Subsequent epoch: 1 day Real time when observations are compared in situ.	Real time and automatic. Telemetric transmission of parameters and results. Alarm property.	In reasonable time when using analytical plotter.	Depends on noise level, cycle slips and experience. It might amount to days when ephemeris are degraded.

The only recent advancements known in the area of geotechnical equipment has been the optical fibre crack-detection and crack-width sensors. These sensors are designed to continuously monitor the formation and sizes of cracks in concrete dams and heavy concrete structures. The theory and the application of the sensors are well described by Haug et al. (1991).

Germany and Canada are the only two countries in this study who have demonstrated the use of sophisticated computational methods for the optimal design and analysis of the geodetic monitoring surveys. For a comprehensive description of the German methodology for the design of control networks the reader is referred to Gruendig (1986). Case studies of existing dams illustrating the applications of least squares adjustment and statistical analysis are given by Bill et al. (1985) and Heck (1985).

Although the Germans can be considered fairly advanced in the area of geodetic deformation networks, they have not yet investigated the possibility of integrating geodetic observations with geotechnical and structural observations.

3.10 ITALY

In 1988, according to the *World Register of Dams* (1988), Italy had officially registered 440 large

dams. Of these 440 dams, there are 97 arch concrete dams, 205 massive gravity concrete dams, 23 buttress concrete dams, 10 multiple-arch concrete dams, 24 rockfill dams and 81 earth dams. A recent figure on the current number of dams could not be obtained since the Italian Committee on Large Dam (ITCOLD) is in the process of updating their dam inventory for inclusion in the next issue of the World Register of Dams (Bonaldi, 1992).

Italy has no national specifications applicable to dam monitoring. The design, construction and operation of dams fall under regulations enacted on November 1, 1959. Since then, the regulations have been integrated into the Technical Regulations dated March 24, 1982. The regulations are general in nature and prescribe the safety rules by which authorities must abide by ICOLD (1989). The design of the measuring system depends upon the dam type, life span, size and the reservoir capacity as well as the possible risk to human life. However, the ITCOLD has recommended general guidelines defining the type of measurements and instruments that should be used to monitor concrete and earthfill type dams. These are given by Tables 3.10a-3.10c (concrete dams) and Tables 3.11a-3.11b (earthfill dams). The guidelines have been included in ICOLD, (1989) as part of the ITCOLD's report. The key requirements of a measuring system are (ICOLD, 1989):

(1) the system should be designed and installed by specialized personnel,
(2) visual inspections should always be made,
(3) redundant observations should be conducted by observing the same points using different instruments,
(4) for automated systems alternatives should be made available to perform the measurements manually, and
(5) there should be a minimum amount of time (next to nil) between the execution of the measurement and the final analysis.

The Legislation commissions the overall supervision of dams to the Dam Service and Civil Engineering Offices of the Technical Department of the Ministry of Public Works. Dam Services and the Civil Engineering Offices play very important roles in all phases of the project. For example, during the operation of the dam, Dam Service and the Civil Engineering Offices are

responsible for carrying out surveillance of the structure. They also visit the dam at least twice a year, during minimum and maximum reservoir levels if possible, and conduct periodic checks of telephone, radio links and other signal and alarm systems (ICOLD, 1989).

Dam safety in Italy is primarily based on the comparison of instrumental data with forecasted data from simple (statistical) models or more sophisticated (deterministic) models. The degree of risk and safety for a dam is established during the design stage. During this stage a reference model is defined. This model establishes the quantities that must be observed and how they are expected to behave. The two major types of models applied to Italian dams are the "a posteriori regressive model (statistical model)" and the "a priori deterministic model." The data from the instruments are independently graphed and compared to the predicted data from one of the two models. For older dams and those with special problems, the current practice is to perform a check-up or a "certified control" every ten years. This may include re-designing the structure to determine the reference model and/or revising the entire measuring system in place and verifying its completeness and adequacy. A complete list of what is reviewed during these visits can be found in the Italian national report in (ICOLD, 1988).

TABLE 3.10 - CONCRETE DAMS IN ITALY
Table 3.10a
Environmental Measurements
(from ICOLD, [1989, p. 156])

Quantity Measured	Construction	Temporary Operation and trial test	Operation	Instruments
Air temp.	*	o	o	Thermometers
Snow & rain fall	o	o	o	Snow & rain gauge
External pressure	o	o	o	Barometers
Humidity	o	o	o	Hygrometers
Water temp.	-	o	o	Thermometers
Water level	-	*	*	Levelling staff, hydrostatic balance

Quantity Measured	Construction	Temporary Operation and trial test	Operation	Instruments
Ice thick.	-	o	o	
Bathometry	-	o	*	Sonic and radar sounding

Table 3.10b
Measurements Within the Dam Body
(from ICOLD, [1989, p. 56])

Qty. to be Measured	Constr.			Temporary Operation and Trial Test			Operation			Instruments
	GR	BU	AR	GR	BU	AR	GR	BU	AR	
Horizontal displacement	-	-	-	**	**	**	**	**	**	Triangulation, collimation, direct and inverted plumb line
Vertical displacement	*	*	-	**	**	*	*	*	o	Topographic or hydrostatic levelling
Rotations	*	*	-	**	**	*	*	*	o	Movable or fixed clinometers
Movements of joints	o	o	o	*	*	*	o	o	o	Dilatometers, joint meters, extensometers
Movements of cracks	**	**	**	**	**	**	**	**	**	Dilatometers, joint gauges, extensometers, acoustic emission
Concrete temp.	*	*	*	o	o	o	o	o	o	Thermometers
Concrete deform.	o	o	o	o	o	o	o	o	o	Extensometers
Stresses	o	o	o	o	o	o	o	o	o	Stress meters
Seepage	-	-	-	-	**	**	**	**	**	Weirs, volumetric measurements

Table 3.10c
Measurements in the Foundation
(from ICOLD, [1989, p. 156])

Qty. to be Measured	Constr.			Temporary Operation and Trial Test			Operation			Instruments
	GR	BU	AR	GR	BU	AR	GR	BU	AR	
Horizontal displacement				**	**	**	**	**	**	Inverted plumb lines, inclinometers
Vertical displacement	*	*	*	**	**	*	*	*	o	Topographic or hydrostatic levelling, extensometers
Rock deformation	o	o	o	o	o	o	o	o	o	Extensometers
Elastic modulus	o	o	o	o	o	o				Sonic core, seismic velocity
Stresses	o	o	o	o	o	o				Stress meters
Under and pore pressures				**	**	*	*	*	o	Pressure cells, stand pipe piezometers, manometric cells
Under-seepage				**	**	**	**	**	**	Weirs, volumetric measurements

TABLE 3.11 - EARTHFILL DAMS IN ITALY
Table 3.11a
Environmental Measurements
(from ICOLD, [1989, p. 157])

Quantity Measured	Construction	Temporary Operation and trial test	Operation	Instruments
Air temp.	o	o	o	Thermometers
Snow & rain fall	o	o	o	Snow & rain gauges
External pressure	o	o	o	Barometers
Humidity	o	o	o	Hygrometers

Quantity Measured	Construction	Temporary Operation and trial test	Operation	Instruments
Water temp.	-	o	o	Thermometers
Water level	-	*	*	Levelling staff, hydro-static balance
Ice thick.	-	o	o	
Bathometry	-	o	*	Sonic and radar sounding

Table 3.11b
Measurements Within the Dam Body and the Foundation
(from ICOLD, [1989, p. 157])

Qty. to be Measured	Construction		Temporary Operation and Trial Test		Operation		Instruments
	UF	C	UF	C	UF	C	
Settlements	**	**	**	**	*	*	Topographic or hydrostatic levelling
Horizontal displacements			**	**	*	*	Triangulation, collimation, inverted plumb line, inclinometers, extensometers, invar wires
Deformations	*	*	*	*	o	o	Extensometers
Total pressures	o	o	o	o	o	o	Pressure gauges
Pore pressure in the dam body		**	**	**	**	**	Pressure cells
Pore pressure and ground water level in the foundation	**	**	**	**	*	*	Pressure cells and stand pipe piezometers
Seepage			**	**	**	**	Weirs, volumetric measurements
Turbidity			o*	*	o	*	Turbidity meters

Notes on Tables:

Relevance of safety levels:
** Critical situation (up to collapse)
* Out of service (total or partial)
o Check

Gr = Gravity dam
BU = Buttress dam
AR = Arch dam
UF = Upstream waterproof
C = Impermeable core

Italians have developed a strategy (ENEL, 1980) for combining the statistic and deterministic modelling in dam deformation analyses. Statistical models (see section 2.3.4) consist of correlations between environmental quantities (e.g., impounded water level, ambient temperatures) and structural effects (e.g., displacements). The correlation is derived from statistical analysis of past data. With the deterministic models (see section 2.3.5), the aim is to derive the effects from the causes by applying known laws of materials and knowledge of the local conditions (e.g., geometry, material properties). The cause quantities are those which induce changes in the structure and the effects are the responses of the structure to the variations of the cause quantities. To improve the analysis, the two models are often used together to form what is known as the hybrid model. Here the knowledge of the expected structural behaviour obtained from the statistical model is used to "tune-up" the deterministic models. Overall, the deterministic and hybrid models are the most often used models for analysing the behaviour of Italian dams (ICOLD, 1989).

The estimated values obtained by the models are taken as the reference values from which the observations (measured effects) are assessed. If the difference between the forecasted effects and the measured effects is within specified tolerance bands, then this confirms the hypothesis that the dam behaves as the model predicted. To visualize what is happening, the measured deformation from an instrument and the predicted deformation from the model are presented and compared in graphical forms (ENEL, 1980).

The tolerance bands are determined from the standard deviation of the forecasted model and the actual measurement. It is estimated on the basis of the data gathered for the normal behaviour of the structure over a period of two to three years. There are three tolerance bands: (1) the first band equalling to twice the standard deviation within which the behaviour of the dam is classified as normal; (2) the second band equalling to three times the standard deviation within which the dam is said to be experiencing slight irregularities; and (3) the third band which is based on the ultimate strength data of the physical and/or mathematical model of the structure causes

suspicions of serious irregularities (ICOLD, 1989). The numerical simulation of the deterministic models are realized by a Finite Element mathematical model. A description of the deterministic model and the application of both types of models are given in the *Comportamento Delle Grandi Dighe Dell'ENEL* (ENEL, 1980).

The Italians assign little importance to geodetic measurements. Their aim is to obtain measurements as quickly as possible and thus their efforts are focused primarily on a system of instrumentation that can be easily automated. The geodetic systems are very simple, comprising a series of plumblines, inclinometers, collimation, bench marks and horizontal survey targets, each analyzed separately. This methodology has limited geodetic measurements and analysis to a one-dimensional case. Examples of the most common types of instruments used in the monitoring system are given at Tables 3.10a-c and 3.11a-b. Some of the recent developments in instrumentations are the Ladir (laser systems for surveying the dynamic displacements of dams) and thermography (systems to study the surface's thermal conditions) (ICOLD, 1989). Some new Italian developments in instrumentations is sonic tomography (sonic log and sonic cross-hole) for diagnosing the state of health of concrete (Bertacchi et al., 1991). Bertacchi et al. (1991) provide a good description of the method and some examples of its application.

Their seismic surveillance system is a complex system consisting of sensors and hardware for acquiring, recording and processing the data. The system is made up of either "distributed" equipment (direct recording using accelerographs or seismographic recorders) or "centralized" equipment (recording the seismic phenomena to a centralized system using communication cables) (ICOLD, 1989).

The major criteria for the frequency of measurements that have been applied by the Italians are listed in Table 3.12. The table has been constructed from the guidelines provided by the ITCOLD's report in the ICOLD, (1989). The frequencies of the measurements depend on factors such as the quantity to be measured, the life span of the structure, the sensitivity of the measuring device, the regulations specified by the authorities and other special requirements (ICOLD, 1989).

Excellent examples of the types of monitoring systems and analysis applied to some of the major Italian dams are included in Bonaldi et al. (1985), Piccinelli et al. (1985), Bonaldi et al. (1984) and the aforementioned *Comportamento Delle Grandi Dighe Dell'ENEL* (ENEL, 1980).

Table 3.12
Measurement Frequencies Applied to Italian Dams
(from ICOLD, [1989, p. 161])

Instruments	Frequency of Measurements
Pendulums, piezometers	Bi-weekly
Inclinometers, strain-gauges, uplift pressure manometers	Monthly
Collimation and settlement devices	Quarterly
Levelling and geodetic measurements	Semi-annually or annually
Ambient quantities	Daily
Seepage	Daily

1. Maximum time between readings of the most critical measurements should not exceed 15 days.
2. For automated measuring systems recording of data may be done daily.
3 During the filling of the reservoir at each stages of the filling , at minimum, one complete series of measurements, and processing and analysis of the data with the design model data should be done.

3.11 JAPAN

Japan has 2,228 large dams which are owned by either the government or private owners (*World Register of Dams*, 1988; Hayashi et al., 1987). There has been a reduction in the number of arch dams built because of the decrease in favourable sites. Consequently mainly rockfill and concrete gravity dams have been constructed, followed by earthfill dams. Japan is well known for the development of the Roller Compacted Dam (RCD) construction method. The RCD construction method reduces the volume of concrete and formwork and uses common equipment such as bulldozers and dump trucks to transport and place the cement. Today, further research is being done to improve the RCD technology to build large dams up to 155 meters high (Nakazawa, 1991).

The Japanese government has taken a fairly conservative approach to dam safety. Dam safety has been achieved through the implementation of studies, the involvement of experts, the careful planning of monitoring systems and the use of specialized construction methods, design criteria and guides. The current regulations regarding the criteria for the design and monitoring of Japanese dams are specified in the following four guides (Hayashi et al., 1987):

1. *Manual of Instrumentation and Monitoring of Dams*, published by the Federation of Electric Power Companies, 1964 (in Japanese).
2. *Criteria of Monitoring and Inspection of Dam Structures*, published by the Japanese Committee on Large Dams (JANCOLD), 1973 (in Japanese).
3. *Guidelines of Design and Monitoring of Dams*, published by the Bureau of Agricultural Construction and the Ministry of Agriculture, Forestry and Fisheries, 1981 (in Japanese).
4. *Guidelines of Monitoring of Safety of Dams*, published by the Public Works Institute and the Ministry of Construction, 1982 (in Japanese).

The effective use of instrumentations and monitoring and the proper analysis of the recorded data are outlined in the regulation guide published by the Public Work Research Institute. The regulations commission the Ministry of Construction as the Safety Administration and Monitoring of Dams. Some of the quantities stipulated in the design criteria are: (1) the permissible pore and uplift pressures in different types of dams and foundations, (2) the allowable stresses due to temperature gradients in concrete dams, (3) the treatment of fractured zones in the foundation rock and the corresponding safety factor, and (4) the seismic coefficients for different types of dams and regions (Hayashi et al., 1987).

Japan places a high priority on the seismic monitoring of dams. The country is located on a very active seismic region. Its dams have suffered from seismic shocks from more than 45 earthquakes between 1943 and 1980. Fortuitous, the damages to the dams have been negligible resulting in minor leakages, settlements and deformations which were repaired immediately (Hayashi et al., 1987). An extensive amount of scientific and theoretical research has been conducted to study the resistance of dams to earthquake. Consequently, whenever new findings are made they are immediately applied to dam design and construction (Kuroda and Baba, 1985).

For example, a recent development has been the electro-magnetic recorders for multi-channel data collection. In addition the systems have been improved by including accelerometers, velocity meters and displacement meters. (Hayashi et al., 1987).

Many experts agree that the design of a monitoring system depends upon several factors including the type of dam, its size and the environment within which it is constructed. Despite this fact there are some common quantities (rotations, displacements, settlements) that are measured within the same classification of dams. For example, JANCOLD has published a very comprehensive table (see Table 3.13) outlining what they believe to be the essential instruments that should be included in the major types of concrete and earthfill dams. This table includes the purpose of the instruments during the construction and maintenance stages. The frequency of monitoring is given by the guidelines reproduced in Tables 3.14a and 3.14b. Included is also Table 3.14c, an actual frequency program that is being implemented on the Miyama Dam located in a high seismic region of Japan (Hasegawa and Kikusawa, 1988). These tables show that the frequency of monitoring is not only a function of time but also a function of the type of dam, seismic activity and flooding. The selection of a monitoring system that is best suited for a dam involves (ICOLD, 1989):

(1) identifying the elements that characterize the safety of the dam and its foundation,
(2) determining the quantities that best describe the behaviour of the dam,
(3) selecting the best instruments for obtaining these quantities,
(4) determining the number and the optimum distribution of instruments, and
(5) determining the frequency of the observations.

The design of automated monitoring systems is based on the advantages and disadvantages of the systems. For example, information from the automated system is very susceptible to corruption by the improper maintenance of monitoring sensors, therefore if automation is to be used, it is essential that experts be included as part of the monitoring requirements. Although automated systems are a consideration, they do not replace visual inspection. As shown in Tables 3.14a and 3.14b, visual inspection of any dam is an essential observation during the first impounding and throughout its life span (Hayashi et al., 1987).

With the exception of surfacing type rockfill dam, Table 3.13 illustrates that Japan's regulations direct that monitoring systems comprise of both geodetic and geotechnical/structural instrumentations. These instruments do not differ significantly from those used by other countries. For any dam, the Japanese regard leakages and deformations to be the most important quantities to monitor and that during the first impounding and after an earthquake it is very important to inspect the dams weak points and its foundation (Hayashi et al., 1987). The surveillance methods for evaluating existing dams is essentially similar to those applied to newly constructed dams (Satake et al., 1985).

According to Hayashi et al. (1987), some of the new monitoring instruments that are still under development include:

1. *Optical Fibre Cable* for transmitting information, a very useful countermeasure against electrical storms.
2. *Ultra Red Laser* for automating distance measurements.
3. *Remote Control Robot System*, used to inspect the dam's gate, pier, aqueduct, foundation rock and slopes.

The observations from the surveillance system are analyzed independently using statistical (regression) analysis based on the progressive history of the structure. The results of the analysis are compared with those predicted from Finite Element Methods (FEM) and presented in graphical forms. In embankment dams, the FEM is also used in analysing the predicted pore water pressure with those calculated with the measured values. Several examples of these analyses are given by Hayashi et al. (1987) and papers written by the members of JANCOLD in the Proceedings of the International Congress on Large Dams (e.g., Fifteenth International Congress on Large Dams, Vol. I, Q.56 (1985); Sixteenth International Congress on Large Dams, Vol. II, Q.61 & Vol. III, Q.62, (1988)).

Table 3.13
Types of Instrumentations Applied to Japan's Concrete and Fill Dams
(from Hayashi et al., [1987, pp.4-5])

			Purpose	Sensor
Concrete Dam	Gravity	Construction	* Control of temperature * Inspection of leakage	Thermometer, Joint meter, Strain meter
		Maintenance	* Deformations of dam and foundation * Inspection of leakage from foundation	* Sighting survey, Plumb line * Piezometric hall, Measuring weir
	Hollow Grav-ity	Construction	* Control of cracking in web plate and diamond head * Deflection	Thermometer, Joint meter, Strain meter, Stress meter
		Maintenance	* Inspection of leakage * Deformation of dam and foundation	Leakage meter, Strain meter, Plumb line, Thermometer, Piezometer hall, Sighting survey
	Arch	Construction	* Deflection and joint movement * Control of temperature * Control of cracking	Thermometer, Joint meter, Strain meter, Stress meter, Plumb line, Sighting survey
			* Observation of rock deformation and leakage around the discontinuous planes	Sighting survey, Plumb line, Strain meter, Stress meter, Rock deformeter, Deflect meter, Measuring weir

<table>
<tr><th colspan="3"></th><th>Purpose</th><th>Sensor</th></tr>
<tr><td rowspan="6">Fill Dam</td><td rowspan="2">Rockfill (Center Core Type)</td><td>Construction</td><td>* Control of excess pore water pressure during embankment
* Control of compaction
* Relative displacement among core, filter and rock abutment</td><td>Pore water pressure gauge,
Soil pressure gauge,
Relative settlement gauge,
Relative shear displacement gauge,
Radio isotope density meter,
Strain meter in inspection gallery</td></tr>
<tr><td>Maintenance</td><td>* Settlement, lateral deformation
* Pore water pressure
* Shear deformation on the abutment, leakage</td><td>Sighting survey,
Strain meter,
Soil pressure meter,
Deflect meter,
Rock deflect meter,
Pore pressure measuring weir,
Shear deformation meter,
Strain meter and deflect meter in gallery</td></tr>
<tr><td rowspan="2">Rockfill (Surfacing Type)</td><td>Construction</td><td>* Inclination and stress in surface pavement
* Soil pressure, pore pressure, settlement, horizontal displacement, compacted density</td><td>Stress meter of reinforcement,
Inclinometer,
Soil pressure gauge,
Pore pressure gauge,
Settlement gauge,
Deflection meter</td></tr>
<tr><td>Maintenance</td><td>* Control of surface cracking
* Leakage, settlement, pore pressure</td><td>Measuring weir,
Settlement gauge,
Pore pressure gauge,
Stress meter of reinforcement</td></tr>
<tr><td rowspan="2">Earthfill</td><td>Construction</td><td>* Soil pressure during embankment
* Pore pressure, settlement of soil layer, lateral displacement, density</td><td>Soil pressure gauge,
Pore pressure gauge,
Piezometer hall,
Settlement meter,
Horizontal displacement meter,
Radio active density meter</td></tr>
<tr><td>Maintenance</td><td>* Leakage
* Settlement, pore pressure
* Lateral deformation</td><td>Sighting survey,
Soil pressure,
Deflectmenter,
Pore pressure gauge,
Measuring weir,
Piezometric hall</td></tr>
</table>

		Purpose	Sensor
Slope around reservoir	Maintenance	* Leakage * Horizontal deformation * Settlement * Inclination	Piezometric hall, Land slide inspector

Table 3.14a
Monitoring Frequency of Japanese Dams
(from Hayashi et al., [1987, p.4])

	Type of dam	QUANTITIES TO BE MONITORED								
		Leakage	Deformation	Uplift or pore pressure	Piezometric height	Temperature	Stress or soil pressure	Strain or shear displacement	Inspection of dam	Seismic observation
First impounding	Gravity dam	1/day	1/day	1/day	-	1/7 days	1/7 days	1/7 days	1/day	Starter sets at about 1 cm/s^2
	Arch dam	1/day	1/day	1/day	-	1/3.5 days	1/7 days	1/7 days	1/day	
	Rock-fill dam	1/day	1/7 days	1/day	-	-	1/7 days	1/7 days	1/day	
	Earth dam	1/day	1/7 days	1/day	1/day	-	1/7 days	1/7 days	1/day	
5 years after impounding	Gravity dam	1/7 days	1/7 days	1/7 days	-	1/ month	1/ month	1/ month	1/ month	Starter sets at about 2 cm/s^2
	Arch dam	1/7 days	1/7 days	1/7 days	-	1/0.5 month	1/ month	1/ month	1/ month	
	Rockfill dam	1/7 days	1/ month	1/7 days	-	-	1/ month	1/ month	1/ month	
	Earth dam	1/7 days	1/ month	1/7 days	1/7 days	-	1/ month	1/ month	1/ month	

	Type of dam	QUANTITIES TO BE MONITORED								
		Lea-kage	Defor-mation	Uplift or pore pres-sure	Piezom etric height	Temp-erature	Stress or soil pres-sure	Strain or shear displ-ace-ment	Inspe-ction of dam	Seismic obser-vation
Long term	Gravity dam	1/ month	1/ month	1/ month	-	-	- month	- month	1/ month	Starter sets at about 5 cm/s²
	Arch dam	1/ month	1/ month	1/ month	-	1/ month	- month	- month	1/ month	
	Rockf-ill dam	1/ month	1/3 months	1/ month	-	-	-	-	1/ month	
	Earth dam	1/ month	1/3 months	1/ month	1/ month	-	-	-	1/ month	

Table 3.14b
Monitoring Frequencies of Japanese Dams
for Specific Cases
(from Hayashi et al., [1987, p. 5])

	Type of dam	QUANTITIES TO BE MONITORED								
		Lea-kage	Defor-mation	Uplift or pore pres-sure	Piezom etric height	Temp-erature	Stress or soil pres-sure	Strain or shear displ-ace-ment	Inspe-ction of dam	Seismic obser-vation
1 Week after flood discha-rge	Gravity dam	1/day	1/day	1/day	-	1/7 days	-	-	1/day	Starter sets at about 1-2 cm/s²
	Arch dam	1/day	1/day	1/day	-	1/7 days	1/7 days	1/7 days	1/day	
	Rock-fill dam	1/day	-	1/day	-	-	-	-	1/day	
	Earth dam	1/day	-	1/day	1/day	-	-	-	1/day	

	Type of dam	QUANTITIES TO BE MONITORED								
		Lea-kage	Defor-mation	Uplift or pore pres-sure	Piezom etric height	Temp-erature	Stress or soil pres-sure	Strain or shear displ-ace-ment	Inspe-ction of dam	Seismic obser-vation
1 Week after seis-mic shook	Gravity dam	1/day	1/day	1/day	-	1/7 days	-	-	1/day	
	Arch dam	1/day	1/day	1/day	-	1/7 days	1/7 days	1/7 days	1/day	
	Rock-fill dam	1/day	-	1/day	-	-	-	-	1/day	
	Earth dam	1/ day	-	1/day	1/day	-	-	-	1/day	

Table 3.14c
Monitoring Frequencies of Miyama Dam in Japan
(from Hasegawa and Kikusawa, [1988, p. 206])

ITEM	INSTRUMENT	FREQUENCY
Uplift	pressure gauge	4 per month
Sedimentation	acoustic method	1 per year
Crest displacement	theodolite	1-8 per year
Joint displacement of membrane	jointmeter	5 per month
Reinforced bar	extensometer	5 per month
Gallery concrete	extensometer	5 per month
Earth pressure	pressure cell	5 per month
Membrane deformation	inclinometer	5 per month
Joint opening along gallery	joint meter and thermometer	daily
Rotation of gallery	inclinometer	3 per month
Leakage	triangle weir (V-notch)	daily

3.12 PORTUGAL

Portugal has 81 large dams of which 71 of these dams are monitored using geodetic and/or geotechnical and structural methods (Florentino, 1992; *World Register of Dams*, 1988). Concrete and masonry dams make up approximately 70% of the total dams, namely gravity, arch, buttress and multiple arch (Pedro et al., 1991).

The dam safety programme in Portugal is governed by the *Dam Safety Regulations 1990 (Decree - Law nº 11/90 6th January, Lisbon)*, and the *Codes of Practice* concerning design, construction, operation and monitoring of dams which is now in the process of being approved by the Portuguese government. These regulations and codes reflect the practice of safety control of Portuguese dams over the past forty years. The regulations assign the responsibility for dam safety to the dam owner and the State Authority the Director General of Natural Resources (Direcção-Geral de Recursos Naturais - DGRN). Furthermore, the regulations stipulate that DGRN, when required, is to be technically assisted by other organizations such as the National Laboratory of Civil Engineering (Laboratório Nacional de Engenharia Civil - LNEC), National Service of Civil Defence (SNPC) and Committee on Dam Safety (Pedro et al., 1991; Florentino 1992; ICOLD, 1988). For example, LNEC may be called upon by DGRN to assist them by preparing or revising the monitoring plans, preparing reports as defined by the monitoring plans and implementing the monitoring data in order to have a continuous representation of the dam's behaviour (ICOLD, 1989). Some of the principal issues of the *Safety Regulations 1990*, primarily those relating to dam monitoring, are discussed in the following paragraphs. However, for a more comprehensive review of the regulations the reader is referred to the Portuguese National Committee on Large Dams' report in ICOLD (1989).

The regulations stipulate that a monitoring program will contain plans for:

(1) visual inspections (e.g., frequency of visits, types of inspections, qualifications of the inspecting agents, main quantities to be observed, format of the report),

(2) installation and operation of the monitoring scheme (e.g., physical quantities to be assessed, specifications of instrument and of their installation and use, frequency of

measurements during the construction, filling and operation phases, qualification of agents charged with the installation and operation of the monitoring scheme, methods of data collecting and processing), and

(3) the methods used to analyze the data and evaluate safety (e.g., behaviour models used to determine the most hazardous scenarios).

The physical quantities monitored and the instrumentation used to monitor Portuguese dams are not significantly different than those practised by other countries. The quantities monitored and the monitoring schemes are summarized in Tables 3.15 - 3.18. Table 3.15 illustrates the quantities measured in concrete and fill dams. Table 3.16 gives information on the placement of the instruments common to concrete and fill dams whereas Tables 3.17 and 3.18 provides information on the placement of the instruments depending on the type of dam (e.g., gravity dam, homogeneous fill dam). Furthermore, the frequency of the measurements are given in Table 3.19. All of the tables reflect the *Dam Safety Regulations* 1990 described in the Portuguese International Committee on Large Dams' report published in ICOLD (1989).

Table 3.15
Summary of Quantities Monitored in Portuguese Dams
(after ICOLD, [1989, pp. 229-234])

Category of Quantities	Physical Quantities	Methods/Instruments Used
CONCRETE DAMS		
External Actions	* Reservoir levels * Air temperature * Water temperature * Ground motion	 * Thermometer * Thermometer

Category of Quantities	Physical Quantities	Methods/Instruments Used
Structural Effects	* Displacements	* Geodetic methods (triangulation, precision traverse and levelling), direct or inverted plumb line, borehole extensometers
	* Rotations * Joint movements * Crack movements	* Electric joint meters, deformeters, electric resistance clinometers
	* Strain * Stress	* Strain meters, no-stress meters * Stress meters, relief methods (bore hole device with stress tension tubes), compensation methods for surface stress (flat jack)
	* Concrete temp. (inside) * Uplift * Seepage	* Thermocouple
Dynamic Effects	* Displacement Velocities Accelerations	* Velocity transducers, microsismographs
	FILL DAMS	
External Actions	* Reservoir levels	Not instrumented
Structural Effects	* Internal/external displacements * Pore pressure * Total pressure * Total flows	* Triangulation methods, precision levelling, USBR cross-arms, slope indicators * Hydraulic piezometers, pore pressure cells * Total pressure cells (Glotzl type hydraulic cells or Maihak type diaphragm cells) * V-notch cells, standard cells
Dynamic Effects	Currently no instruments have been installed to observe dynamic phenomena.	

Table 3.16
Portuguese Monitoring Schemes Common to Concrete and Earthfill Dams (after ICOLD, [1989, pp. 229-235])

Phases	Common to Concrete Dams		Common to Fill Dams	
	Methods and Instruments Used		Methods and Instruments Used	
Construction	* Thermometer (or other electric resistance instruments)	* Provide info to possibly correct construction procedures (lift heights, concrete intervals, artificial cooling).	* Piezometers	* In the foundation and earth zones.
	* Rockmeters	* Placed under concrete block to allow the assessing of the foundation deformability.	* Total pressure	* In the interface zones between materials of different deformabilities.
	* Creep cells	* Placed inside concrete to provide the evolution of the deformability of the concrete and data to determine the stress for the strain meters.		
	* Embedded strain and stress meters	* Provide the reference state of stress of the dam foundation and structure at the end of the construction period.		
First Filling	* Horizontal displacement	* By means of plumb lines and at certain reservoir levels by geodetic methods.	They are similar, however it depends upon the type of fill dam (see Table 3.18).	
	* Vertical displacement	* Precision levelling or rockmeters.		
	* Joint movements	* Deformeters.		
	* Displacement of dam foundation	* Rockmeters.		
	* Seepage and uplift			

Phases	Common to Concrete Dams	Common to Fill Dams
Operation	Measure quantities related to safety and those mentioned in the construction and filling phases.	

In the field of automation, Portugal has long recognized the need for continuous data recording for certain instruments like plumb lines, thus some recording instruments have been used in concrete dams. However, owing that Portuguese dams are accessible throughout the year and that the necessary automation equipment is readily available off-the-shelf, the priority for the development of electronics has been to fulfil rapid data processing and efficient behaviour analysis. With respect to fill dams automatic data acquisition has not yet been considered, but the automatic processing for data analysis is similar to that used for concrete dams (ICOLD, 1989). An example of a typical data processing and analysis system is given by Florentino et al. (1985).

The analysis of the data is performed in stages. In the first stage the data is inputted into the computer where it is checked for major errors and converted into the geometrical and physical investigated quantities (stress, strain and deflection). These quantities are further validated by means of a model analysis and then plotted by subroutines in tabular or graphical forms in order to interpret the behaviour of the dam (e.g., diagrams of the distribution of displacements, stresses, uplifts, seepage and temperatures). In the second stage these results combined with the information gathered from visual inspections and existing documents are further classified by either a qualitative or quantitative method.

Table 3.17
Portuguese Monitoring Schemes Particular to the Major Types of Concrete Dams (after ICOLD, [1989, pp. 229-234])

Quantities	Instruments and Method Used	
Arch Dams		
* External Actions & Ground Motion		* No permanent instrumentation has yet been installed. Studies have been made to install them in existing and future dams.
* Planimetric Displacement	* Triangulation	* Targets placed over the body of the dam to obtain a good representation of the global movement. Along arches usually at two levels (one near the crest and other at mid height or even at more levels when justified). Along cantilevers (one along the cantilever or three or more distributed on the downstream side of the dam body). Precision traverse (as mentioned in Table 3.15).
	* Plumb Lines	* Installed with geodetic targets as a check.
	* Borehole Extensometers	* To obtain absolute displacements of the intermediate anchored points and the rotations of the corresponding sections. In the vicinity of coordimeters near the foundation to obtain the vertical component of the movement of these points. Also crossing significant foundation cracks or faults where movement may effect the behaviour of the dam.
* Altimetric Displacement	* Precision Levelling	* Points located at dam crest and when feasible at inspection galleries and downstream surroundings.
	* Invar Wires and Rockmeters	* Connecting galleries by means of connecting invar wires with rockmeters anchored to deep point installed galleries near the abutments.
* Hydraulic Behaviour	* Piezometers and Drain Curtains	* To measure seepage and uplift pressures.
* Joint Movements	* Deformeters	* Practically in all joints in accessible zones (e.g., on the crest, near dam face, inside inspection and drainage galleries).
* Movement of Points Inside the Body of Dam	* Jointmeters	* In the medium zones of the block or at a distance of about one meter to the dam face and according to the information needed.
* Rotations	* Electric Clinometers	* Scarcely measured but there is a trend to measure them again.
* Concrete Temperature	* Thermometers	* As all other electric resistance instruments (stress and strain meters) give values of temperature, resistance thermometers are installed in a supplementary way. To obtain the information about the mean temperature and the thermal gradient variations along some strategic selected sections.
* Strain/Stress	* Strain/Stress Meters	* Placed in zones where the highest stress is anticipated (i.e., along the arch crest and the main cantilever and near the foundation haunches).

Quantities	Instruments and Method Used	
	Gravity Dams	
* Planimetric Displacement	* Triangulation	* targets over the body to obtain movements of the different blocks, especially higher ones and those located at different deformability zones. Where possible at each block at least one near the crest and the other near the foundation. Also depending on the dam height one or more intermediate markings may have to be installed.
	* Precision Traverse	* Installed in some horizontal galleries.
	* Plumb Lines	* Installed in one or more blocks.
* Vertical Displacement	* Precision Levelling	* At the dam crest, in some cases in horizontal galleries and drainage galleries and in galleries near the foundation.
* Joint Movements	* Deformeters	* Used to monitor relative movements between adjacent blocks usually installed in places similar to those of arch dams.
	* Electric Joint Meters	* Inside joints.
* Uplift Pressures		* More important then arch dams. Special attention is given to the drained water turbidity.
* Rotations	* Electric Clinometers	* Like the arch dams these measurements are just being measured again.
* Temperature	* Thermometer	* Internal temperature in sections distributed along the height of the block. One or a small number of blocks need to be instrumented because thick concrete has a very high thermal inertia.
* Stress, Strain	* Strain Meters	* Usually installed in high dams only. The usual low stress developed inside gravity dams in most cases does not justify its measurement.
	Buttress Dams	

Quantities	Instruments and Method Used	
* External Actions & Ground Motion		* When justified. Only one dam (Aguieira Dam) has been installed with permanent equipment to monitor this phenomena.
* Planimetric Displacement	* Special Triangulation Networks	* The geometric characteristics of the buttress dam calls for a special network. Targets are distributed over the body of the dam buttress and arches at different levels.
	* Plumb Lines	* To measure displacements of points near the foundation and of points of the arches and buttresses. Special installation may be required, for instance plumb lines incased in external devices.
* Vertical Displacement	* Precision Levelling	* Bench marks placed at the crest of the dam and along inspection galleries and downstream zones.
	* Rockmeters	* They may be required to measure foundation movements in some particular zones.
* Internal Joint Movements	* Carlson Apparatus	* In joints of the arches and, at several elevations, in some joints near the dam faces.
	* Deformeters	* Surface joint movement in accessible zones.
* Hydraulic Foundation Behaviour	* Piezometers and Drainage Curtains	* Their placement depends upon the foundation characteristic and the development of water tightness and drainage works during the construction phase.
* Temperature	* Thermometers	* Due to the small thickness of the arches the temperature variations are considerable.
* Strain/Stress	* Strain/Stress Meters and Creep Cells	* Installed in the most typical zones.
Solid Buttress Dams		
* Planimetric Displacement	* Triangulation Networks	* Targets placed over the body of the dam namely near the crest and at one or more levels on the buttresses downstream faces.
	* Plumb Lines	* To measure displacements of some points near the crest or of points near the foundation. Plumb lines encased in external devices are used to measure the displacement of points that cannot be monitored using solely plumb lines.
* Vertical Displacement	* Precision Levelling	* Bench marks placed at the dam crest and sometime at the downstream zones.
	* Rockmeters	* They have just recently been considered.
* Internal Joint Movement	* Joint Meters	* Installed at half-thickness between the heads of the blocks.
	* Surface Joint Deformeters	* Measure opening and sliding movements of joints at accessible zones.
* Concrete Temperature	* Thermometer	* The differences in thickness between the head and the web of the buttress may cause peculiarities in the temperature pattern inside the concrete.
* Strain/Stress	* Strain/Stress Meters	* Groups of each together placed in the most typical zones of the body of the dam may be very useful in the behaviour analysis.

Table 3.18
Portuguese Monitoring Schemes Particular to the Major Types of Fill Dams (after ICOLD, [1989, pp. 234-235])

Quantities	Instruments and Method Used	
	Homogeneous Dams	
* Surface Displacement	* Triangulation * Precision Levelling	* Surface monuments are distributed such that they can be used to model the entire surface of the dam. * Bench marks are usually placed at the crest (if vertical movements are measured) or sometimes at the berm.
* Internal Displacement		* Used to determine the distribution of the horizontal and vertical displacement.
* Pore Pressure		* Measured in one or more sections usually the highest section. In the transition Zones between the body of the dam and the abutments.
* Water Seepage		* Where drainage galleries exit or when devices collection total or partial that of water are available.
	Zoned Dams	
* Surface/Internal Displacement		* Identical to those of homogeneous dams. Some internal displacement devices are placed in the transition zones between different material.
* Pore Pressure	* Pore Pressure Cells and Piezometers	* In clay cores, they are placed in one or more of the highest sections. Further to these sections piezometers may be placed in other sections that are considered important for the dam safety. They are also placed in transition zones between the dam and the abutments.
* Total Pressure	* Piezometers	* Placed in clay cores and in transition zones between different materials. Their distribution is important in the safety evaluation with respect to hydraulic fracturing.
* Seepage		* Same consideration as for homogeneous dams.
	Rockfill Dams With Upstream Impervious Face	
* Displacements	* Triangulation and Precision Levelling Methods	* Survey monuments and bench marks are placed on the crest and on the downstream face. Consideration should be made to measure displacement of points on the upstream face in some sections by means of slope indicators and measurement of displacement along transverse and longitudinal lines.
* Flows	* Pressure Cells and Piezometers	* Partial and total flows at the upstream face since the impermeability is only guaranteed at the upstream face. **Note: The remaining of the quantities measured are similar to those of the other dams described in this table.**

Table 3.19
Frequency of Measurement of Portuguese Dams
(after ICOLD, [1989, pp. 235-236])

Phases	Instruments and/or Quantities Measured	Frequency of Measurements
Concrete Dams		
During Constructions	* Electrically Embedded Apparatus	* During installation - every 4 hrs until 12 hrs after. After installation (24 hrs after) - every day at on the same hour for the first week. After one month of installation - every 2 weeks. Following period - once a week.
During Filling	* Visual Inspection * Measurements of Displacement by Geodetic Methods	* Continuous. * To achieve a rapid safety assessment especially when the water reaches certain levels of the reservoir, at such time an analysis must determine a thorough understanding of the behaviour of the dam.
After First Filling	* For the first 5 years * For the subsequent years	* For essential quantities - weekly Displacements by geodetic methods - annually For the remaining quantities - fortnightly * The frequency of the measurements usually decrease, however displacements by means of plumb lines and borehole extensometers and seepage and uplifts are still measured once a week.
Fill Dams		
During Construction	* Pore/Internal Pressure	* Measurements are made immediately after installation of the instruments as soon as conditions allow it and their frequency depends on the construction development.
First Filling		* As stipulate in the specifications in the monitoring plans.
After First Filling	* Measurements and Visual Inspections	* Under normal circumstances (dam behaves as expected) the measurement are conducted every 2 years. This will be changed by the new Dam Safety Regulations.

The qualitative method consists of detecting the correlation between the actions, mainly water level and temperature, and the corresponding physical quantities as those given by the diagrams. In some cases the correlation is tempted using mathematical models such as hydraulic, thermal and structural models (Pedro et al., 1979; ICOLD, 1988). On the other hand, the quantitative method consists of setting up a quantitative model (statistical, deterministic or hybrid type) which

describes the functional relationship between the observed effects and the corresponding actions. The model attempts to predict the values of some physical quantities, taking into account factors including material properties, geometric characteristics and previous behaviour of the dam (ICOLD 1988; Gomes and Matos 1985). Some examples of the different types of quantitative models that have been applied to existing dams are given by Gomes and Matos (1985).

Additional information regarding the different types of monitoring systems and data analysis addressed above, complete with case studies, can be found in articles by Ferreira de Silva et al. (1991), Pedro et al. (1991), Florentino et al. (1985), Gomes and Matos (1985), Ramos and Soares de Pinho (1985), Casaca (1984) and Pedro et al. (1979).

3.13 SOUTH AFRICA

The first dam constructed in South Africa (SA) was a large (h = 15 m) U-shape embankment dam in 1652 (Olwage and Oosthuizen, 1985). Today, three centuries later, there are over 452 large dams registered in SA: 257 (57%) embankment dams and 199 (43%) concrete dams (*World Register of Dams*, 1988).

In SA, most large dams are owned and engineered by or on the behalf of the SA Department of Water Affairs (DWA). Dam safety in SA originated with DWA in 1978 when safety inspections of dams owned by DWA were conducted every 5 years by a competent team of engineers. A giant leap towards the implementation of safety regulations occurred in 1982 when a Dam Safety Directorate was established in the department of DWA to develop a dam safety programme (Oosthuizen, 1984). However, it was not until recently, in 1986, that the South African Government introduced Dam Safety Regulations. The regulations insure that plans for monitoring systems for new dams are approved by the appropriate authority and that the adequacy of monitoring systems of existing dams are evaluated every 5 years (ICOLD, 1989).

The majority of the documents related to dam safety in SA have been written by the DWA

personnel who are also active members of the South African National Committee on Large Dams (SANCOLD). Consequently, the facts reported in this discussion reflect solely the views, practices and experiences of DWA.

There are no specifications for dam monitoring in SA. Attempts by DWA to write specifications failed due to the multiple restrictions and limitations associated with existing dams and the requirements of a monitoring system peculiar to a site with new dams (ICOLD, 1989). As an alternative to specifications, DWA has developed an underlining philosophy for the design of a monitoring system for a particular site. This philosophy dictates the basic factors which DWA believers should be considered when designing an optimum monitoring system. These include:

(1) the function of the instrument system,
(2) the phase in the life of the dam for which the system is required (pre-construction, construction, initial filling, normal operation, etc.),
(3) the physical conditions of the dam (geology, design assumptions, hazard potential, etc.), and
(4) the particular operational conditions (e.g., the ability and attitude of the observer).

On the basis of this philosophy, DWA recommends a series of steps that should be followed during the design of a monitoring system (see Table 3.20). Furthermore, DWA often refers to ICOLD Bulletin N° 41 as a guide for determining the initial parameters to be measured and the required frequencies of the measurements (ICOLD, 1989).

Table 3.20
Steps for the Design and Installation of a Monitoring System as Suggested by the South African Department of Water Affairs (after ICOLD, [1989, p. 252])

<table>
<tr><td>1. Determine Function(s) and Monitoring Phases</td><td rowspan="2">• Determine site conditions
• Identify critical and key aspects</td><td rowspan="2">Study the expected behaviour of the structure by determining:
• the loads acting on it
• its capacity to resist these loads
• other factors influencing the response of the structure (design redundancies, etc.)</td></tr>
<tr><td>2. Study Similar Systems</td></tr>
<tr><td>3. Preliminary Design</td><td colspan="2">• Identify parameter to be measured.
• Select location and number of instruments.
• Select read-out frequencies.
• Determine required accuracy, measuring range, sensitivity, repeatability of the various instruments.
• Select types of instruments, sensor read-out units, etc.
• Tailor lay-out to acceptable limits.
• Prepare preliminary drawings and documentation.</td></tr>
<tr><td>4. Intermediate Evaluation</td><td colspan="2">• Consult Designer, Dam Safety Specialists, Suppliers, Construction Team.
• Perform a cost-benefit analysis based on the alternatives. Aspects such as installation cost, and costs (and convenience) for taking readings and processing the data in the long term, should also be included.</td></tr>
<tr><td>5. Final Design</td><td colspan="2">• Prepare scenarios for instrumentation failures and design and specify defensive back-up systems.
• Finalize design (simplest arrangement of devices suitable for the purpose).
• Write specifications (tenders, installation, etc.)</td></tr>
<tr><td>6. Installation</td><td colspan="2">• Obtain co-operation of construction staff on site.
• Plan the installation procedure in detail.
• Make final preparations for installation (logistics, re-calibration, etc.)
• Install or supervise installation.
• Keep good photographic records.
• Record design adjustments.
• Do performance checks.</td></tr>
</table>

7. Final Report	Prepare an installation report containing: • As-built drawings. • Zero readings. • Records (photographs). • Instructions and manual to the observer and analyst. • An evaluation of the installation.

The monitoring instruments and practices preferred by DWA are summarized in Table 3.21. SA has taken a rather simplistic approach to dam monitoring. In SA, most dams are situated in remote areas hundreds of kilometers away from the organization responsible for dam safety evaluation and therefore, authorities do not considered it feasible to have sophisticated equipment on site which requires specialized technicians to operate and maintain. Overall, DWA prefers geodetic methods to monitor external absolute displacements, and geotechnical/structural techniques for relative internal deformations. Where applicable, in the case of concrete dams, traverse methods through galleries and drainage tunnels are preferred over triangulation of points on the external surface of the dam (ICOLD, 1989). This decision was based on the results of a case study by Roberts et al. (1985). Roberts et al. (1985) compared the triangulation method with the traverse method on the bases of accuracy, man power requirements, time and cost constraints, and suitability under all operational and weather conditions. The case study concluded that the advantages of the traverse method far outweighs those of the triangulation (see Roberts, 1985).

Table 3.21
South Africa's Preferred Instrumentation and Methods for Monitoring Dams
(after ICOLD, [1989, pp. 245-247])

Dam Type	Quantity Measured	Methods Instruments	Remarks
		Displacements	
Concrete	Relative	• 3-D displacement crack monitoring devices (e.g., Wexham crack width meter accurate to ± 0.02 mm or ID crack width meter accurate to ± 0.1 mm • mechanical type pendulums with optical reading • tiltmeters	• the ID crack meter was developed by DWA, designed primarily for use during inspections
	Absolute	• geodetic surveys (precise levelling and triangulation, traverse in galleries and drainage tunnels)	• for many existing dams this method is the only practical solution
Embankment	Internal Settl.	• along lines in transverse directions a group of hydraulic settlement cells are used • along the vertical axis magnetic rings are installed around inclinometer tubes at fixed levels	• (i.e. along cross-sections) • these are installed during constr.
	External (on embank. or berm)	• levelling and triangulation	• for measuring settlements and other relative movements
	Inclin.	• inclinometers with torpedo- shaped sensors containing sensitive sensors	
		Pressure	
All Types of Dams	Pore Pressure	• stand pipes and twin tube hydraulic piezometers • resistance and piezo-electrical types • hydraulic membrane type of piezometers	• used to measure piezometric level • are recommended for short term measurements
	Earth Pressure	• Bourdon type of pressure meters • hydraulic type cell with transducers	• for use with the hydraulic type piezometers • most favoured (can be calibrated in situ)
		Strain and Stress	

Dam Type	Quantity Measured	Methods Instruments	Remarks
	Strain	• sliding micrometers	• preferred over any other borehole extensometers • has an accuracy of better than 10^{-3} mm/m
	Stress	• small diameter uniaxial stress meters • smaller bi-axial version of a similar stress meter as the uniaxial one	• it was still being tested by DWA at the time this table was published

As part of the monitoring program, DWA considers site inspection by a trained inspector to be indispensable in the process of establishing the complete behaviour of the dam. DWA conducts quarterly inspections in additional to the frequent inspections by local dam owners and periodic dam safety inspections by experienced personnel (Croucamp, 1985).

Regarding data analysis, DWA has a central data processing and evaluation center for most of its dams. DWA fully recognizes the disadvantages of this method and has taken steps to equip some of its personnel with programmable pocket calculators to take the readings and compare them with those obtained from the behaviour model at the central processing station. In addition, DWA's goal is to have a semi-automated system using a microcomputer at the dam site so that local observers can process the readings and make a reasonable interpretation of the behaviour of the dam expediently and without having to wait for the results from the central office. As with the pocket calculator, these results can then be compared to those of the behaviour model (ICOLD, 1989).

With regards to dam monitoring DWA professes that it has gained a considerable amount of experience and therefore, has taken a number of precautionary steps (referred to as the *golden rules).* Some of these include:

1. When designing an instrumentation system it is necessary to consider all of the possible

mode of failure and malfunctions of the system.

2. It is not only good practice to use a back-up system but it is also advisable to install a *new* type of instrument of unknown performance along side a proven instrument in order to gain experience and confidence of the new instrument.
3. All instruments must be recalibrated, preferably at the same altitude and under the same conditions as those where the instruments are to be used.
4. It is a good practice to study the operating principles and design of each instruments carefully.
5. It is a good practice to use instruments with a good performance record.
6. Lightning protection must be considered (this is something which is often neglected).
7. Inspections by local operators and his staff, apart from the other inspections, is one the most valuable method of monitoring the behaviour and condition of the dam.

Additional examples reflecting the types of monitoring systems and data analysis used in SA can be found in a number of case studies reported by members of the SANCOLD at the ICOLD Fifteenth International Congress on Large Dams, Q56, (e.g., Croucamp 1985; O'Connor 1985; Melvill 1985; Van Der Spuy et al. 1985).

3.14 SWITZERLAND

Switzerland has officially listed approximately 144 large dams, 95% of which are monitored using geodetic instrumentation and 100% are monitored using geodetic and/or geotechnical/structural instrumentation (Bischof, 1992). In Switzerland there are no detailed national specifications defining the monitoring of dams, the types of instruments required and the frequency of measurements (ICOLD, 1989). The safety of dams is governed by the federal law through the *Dam Regulations Act* enacted on July 9, 1957. These regulations contain precise procedural rules and not technical requirements and thus guarantee a high degree of flexibility, allowing the supervising authority to decide and enforce the safety standards according to the current technology and the specific circumstances (Biedermann, 1988). A copy of the legisla-

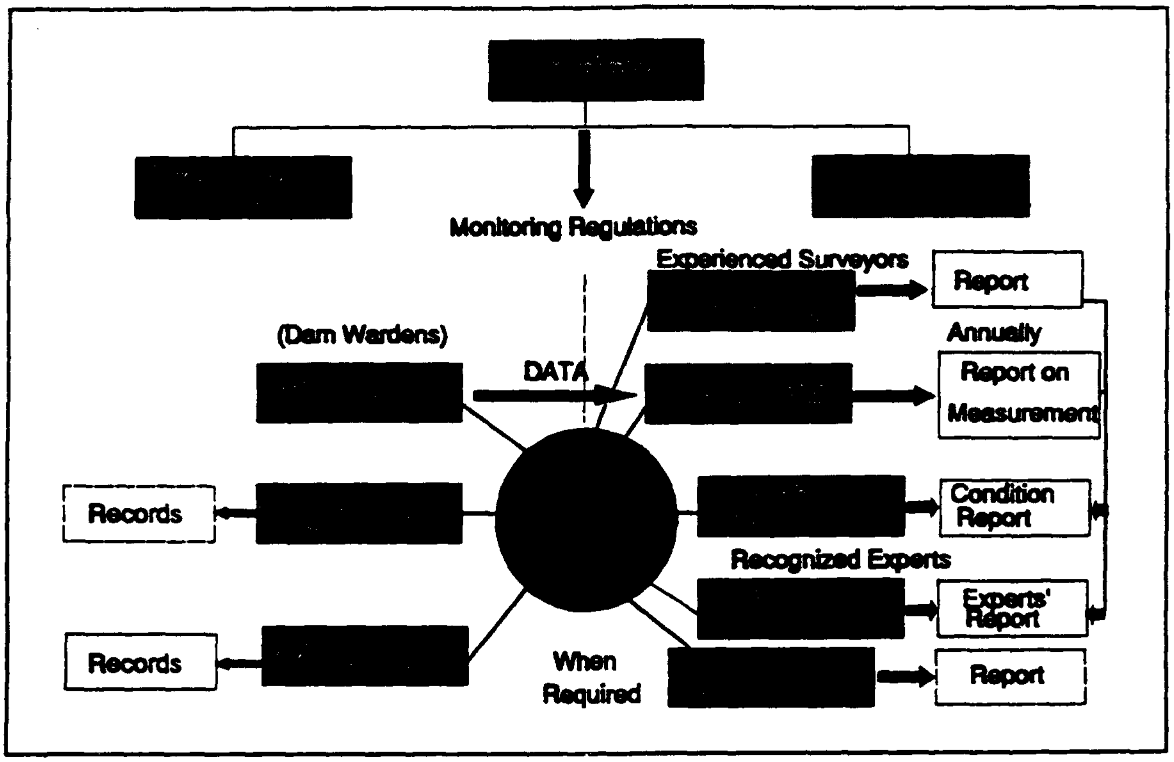
tions and a sample specifications from Swiss Dams: Monitoring and Maintenance are attached as Appendix 11. The specifications were provided by the Swiss National Committee on Large Dams (SNCLD) at the Sixteenth International Congress on Large Dams (Biedermann, 1988).

As a result of this policy, the safety of dams is the responsibility of their owners. They are free to select what they consider to be the most suitable equipment and monitoring methods, all of which is subject to approval by the Swiss Federal Surveillance Authorities (ICOLD, 1989). The reader is referred to the ICOLD, (1989) for a list of duties and responsibilities of the dam owners to the surveillance authorities. The Swiss Federal Surveillance Authorities supervise the execution of monitoring and carry out critical analysis of reports submitted to them on the condition and behaviour of the dam. These authorities also participate every two-to-three years in annual inspection visits and always attend the five yearly inspections undertaken by experts.

The methodology used in monitoring Swiss dams is illustrated in Figure 11 (Biedermann, 1988). One of the essential ingredients of Switzerland's dam safety program is a monitoring system comprised of suitable instrumentation, rapid analysis, and interpretation of readings (ICOLD, 1989). Monitoring itself consists of measurements from the instrumentation system and visual inspection, neither being sufficient on their own. Like Argentineans, the Swiss also believe that a monitoring system should be sufficiently redundant. The Swiss try to achieve redundancy by maintaining parallel but separate sets of instruments and, in addition, facilities for evaluating data by double-checking and by using alternative measurement methods (e.g., plumb line & traverse, alignment & triangulation, and settlement gauge & levelling) (Biedermann et al, 1988).

Switzerland is one of the few countries in the world that believe or admit that supervision of large structures such as dams is today an interdisciplinary task of Civil Engineers, Surveying Engineers, and Geologists, and that a mutual understanding amongst them is essential (Gilg et al, 1985). All Swiss dams are monitored using either geodetic or both geodetic and geotechnical and structural instrumentations. When compared to other countries, Switzerland places more importance on geodetic measurements. Geodetic systems are regarded as complementary control tools. They are based on a horizontal network (angular alignments, triangulation, traverse, optical

Figure 11 Monitoring of Swiss Dams
(after Biedermann et al eds., [1985, p. 35])



alignment, EDM) connected to a vertical network (pendulum, levelling). Geodetic schemes are closely coordinated by specialists such as Civil and Surveying Engineers. The Civil Engineer specifies which deformations at which points should be measured and the Surveying Engineer specifies how to best coordinate the various geodetic methods. These methods are used for both short- and long-term monitoring. A new method which is also under consideration for determining the absolute displacements is the Global Positioning System (GPS) (Egger and Schneider, 1988).

Automated monitoring systems are not considered imperative but are somewhat convenient. They are often used when the dam site is not readily accessible (e.g., dams located in high mountainous regions which are not always accessible in winter) or simply as an extension to the surveillance system. The Swiss strongly believe that one cannot depend completely on the reliability of the data from an automated measuring system. An automated system must never replace the classical monitoring methods (Biedermann, 1988; ICOLD, 1989).

The ICOLD (1989), Biedermann et al. eds. (1985) and other publications published by SNCLD/ICOLD show that the geotechnical and structural instrumentation installed in Swiss dams are not unlike those installed in dams of other countries. The major difference is that Switzerland's Safety Regulations force dam owners and authorities to ensure that the dams are monitored and analyzed with the most recent equipment and methods available. Examples of the types of instrumentation used to monitor and analyze both concrete and embankment dams are provided in the sample specifications (see Appendix 11).

The methods of analysis consist of graphical representations including one or more of the following: (1) a graph of the results in a chronological representation (used when the seasons have a much greater influence than reservoir levels); (2) a graphical representation of variables such as displacement and seepage as a function of the reservoir level (used when the water levels have a predominant influence); and (3) a graphical representation based on "equal conditions" (used when the measured results are corrected for hydrostatic and seasonal effects). The latter method, known as the "compensated displacement" method, is applied above all of the rest for determining upstream-downstream displacements of concrete dam crest and sometimes for other points on vertical dam profiles (ICOLD, 1989). For evaluating geodetic schemes with redundant observations, mathematical models using the least square adjustment are used. If there is doubt as to whether or not the reference points are fixed, a more sophisticated method such as the Helmert transformation is used (Egger and Schneider, 1988).

Deterministic, statistical or combined (hybrid) methods are being increasingly used to calculate the deformations for a standard loading (e.g., water load) or to convert the measured deformations into the deformations that would result from a standard loading. In the first case, the differences in the deformations between those obtained by one of the graphical methods (measured) and those calculated are compared to an acceptable margin of error. In the second case, the measured displacements are used as displacement inputs and if the dam behaves as expected the same deformation values will be obtained (Biedermann et al. eds., 1988). An excellent source which provides numerous examples of the surveillance of Swiss dams is entitled *Swiss Dams: Monitoring and Maintenance* (Biedermann et al. eds., 1988). More examples of the Swiss

approach to dam monitoring are given by Gilg et al. (1985) and Kovari (1985).

3.15 UK

Great Britain is one of the first country to enforce government regulations concerning dam safety. The current act is the *Reservoir Act (RA) of 1975*. This act surpasses the initial act, the *Reservoir (Safety Provisions) Act of 1930*, in that it is much more stringent and effective (Penman, 1982). The 1975 RA does not apply to the province of Ireland (Millmore and Charles, 1988). The current legislation applies to reservoirs which hold or capable of holding 25,000 cubic meters of water above the natural level of any part of the land adjoining the reservoir (Charles and Tedd, 1991). There are about 2,450 dams that come within the scope of the 1975 RA (Hinks and Charles, 1992). However, in accordance with the ICOLD definition of "large dams" only about 529 of the 2,450 dams in the UK are considered to be large dams. Furthermore, of these 529 large dams approximately 408 are embankment type dams and the remaining 121 are concrete type dams (World Register of Dams, 1984).

The provisions of the 1975 RA are described by Johnson et al. (1979). In summary, under the act:

(1) there is a requirement for all reservoirs to be registered and kept under continual supervision by a qualified Civil Engineer,
(2) the power of supervision is assigned to the Secretary of State,
(3) there is a provision for establishing enforcement authorities, and
(4) conditions are outlined for criminal liabilities and other administrative procedures (e.g., remedial measures are taken if the owner fails to respond to the deficiencies of the inspecting engineer's report).

The 1975 RA was not implemented until 1986. Before the act was implemented it was amended by the Scottish Hydro-Electric Board. One significant change was converting the reservoir surveillance and instrumentation policy from a system of manual recording and monitoring to one with more automated equipment and computer software. Another notable change was the

addition of the clause stating that every large reservoirs has to have a Supervisor Engineer with the appropriate experience and who is a member of the Supervising Engineering Panel (Beak, 1992).

The responsibility for implementing the reservoir safety legislation act has been given to the Department of the Environment (DOE). Within the scope of the act, DOE organizes research programmes dealing with the safety of reservoirs. Furthermore, the Building Research Establishment (BRE), a sub-department of DOE, conducts research into the safety of embankment dams (Charles and Tedd, 1991).

Some of the outcomes of BRE's research has been the development of new types of instruments and methods for monitoring and analysing embankment dams. One development has been the

$$S_I=\frac{s}{1000.H.\log(t_2/t_1)} \tag{3.1}$$

so called settlement index (Eq 3.1). The index is used to interpret the results of crest settlements in order to assess the performance of embankment dams. In equation 3.1, s is the crest settlement in millimetres measured between time t_1 and t_2 after the completion of the embankment construction, and H is the height of the dam in meters. Some of the findings suggested from a review of the settlement indices calculated for a number of British dams, based on the data collected over a period of eight to ten years, are that the settlement pattern is affected by reservoir operation and in particular by major draw downs. Another major development has been the simple index ratio $(\sigma_{ha}/\gamma_w h_w)$ which can be used as an indicator of the susceptibility to hydraulic fracture. The numerator of the index ratio, σ_{ha} ,is the total horizontal stress acting in the direction along the axis of the dam and the denominator, $\gamma_w h_w$,is the full reservoir pressure at the depth of the measurement. If the index ratio is greater than unity then hydraulic

fracture is unlikely to occur (Charles and Tedd, 1991). Other examples of such developments are given by Charles and Tedd (1991).

BRE's work is often published for use by engineers in carrying out safety evaluations under the 1975 RA. For example, BRE is the author of the UK *Engineering Guide to the Safety of Embankment Dams (Building Research Establishment Report BR 171)*, published in 1990, and the co-author of the *Engineering Guide to Seismic Risk to Dams in the United Kingdom* (Charles and Tedd, 1991; Hinks and Charles, 1992).

British dam experts believe that the continuing safety of an embankment dam largely depends on visual surveillance supplemented by the installation and monitoring of instrumentation. These experts also consider pore pressures, seepage and settlements to be the most helpful indicators of the behaviour of embankment dams (Charles et al., 1992).

A sample of the types of instruments and frequency adopted by the UK are shown in Table 3.22. Table 3.22 illustrates the policy developed and adopted by the Northern Scotland Hydro-Electric Board.

Table 3.22
Types of Monitoring and Frequency
Adopted by Scotland Hydro-Electric Board (UK)
(from Johnson et al., [1979, p. 251])

Type of Dam	Condition and Behaviour	Age of Dam	Type of Monitoring and Frequency
Arch	Normal	First five years	Full instrumentation quarterly
	Normal	After five years	Full instrumentation twice a year
Gravity and Buttress	Normal	First five years	Full instrumentation twice a year
	Normal	After five years	Full instrumentation for one cycle every fifth year

Type of Dam	Condition and Behaviour	Age of Dam	Type of Monitoring and Frequency
Embankment (Earth or Rockfill)	Normal	First five years	Levelling twice a year. Pore pressure monthly (earthfill only).
	Normal	After five years	Levelling once a year. Pore pressure twice a year (earthfill only).
Gravity and Buttress	Unexpected or uncertain behaviour or where special circumstances apply	First five years	Full instrumentation twice a year. Pendulums four times a year (if installed).
	As above	After five years	Full instrumentation for one cycle every 3-5 years or as considered necessary. Pendulums four times a year (if installed) otherwise instrumentation of key stations twice a year.
Embankment (Earth or Rockfill)	As above	All ages	Levelling twice a year. Pore pressures monthly (earthfill only)

Note: (1) As the dam becomes older or deteriorates, the frequency of the measurements may be required to increase.

(2) If the dam is built in doubtful conditions or with abnormal behaviour, more extensive instrumentation is installed and the frequency of the readings are increased.

(3) For arch dams, and large dams subject to large variations in water levels, more frequent cycles are carried out.

(4) Supervisory inspections and monitoring are undertaken after major floods, seismic activity or unusual events.

The current policy for instrumenting and monitoring dams in the UK is well described by Beak (1992). Generally, Beak (1992) indicates that the trend in the UK has been to replace the typical geodetic, geotechnical and structural instrumentations (e.g., crest levelling, alignment using a collimator, pendulums, crack and joint measurements using callipers, temperature sensors cast into the concrete, piezometers) with new techniques that are more adaptable to automation (e.g., automatic pendulums to measure movement in both directions, displacement transducers to measure cracks and joints, thermocuples for concrete temperatures, invar wires to monitor longitudinal movements in dam galleries, vibrating wire strain gauges attached to concrete

surfaces to enable strain measurements in a plane). The goal is to have all of the readings recorded and the information transmitted through a land line to a remote computer. Articles written by Beak (1992), Charles et al. (1992), Penman and Kennard (1982) and Penman (1982) give excellent examples of the types of instruments and monitoring systems applied to dams in the UK. Included in some of the articles are detailed descriptions and drawings of the instruments used, as well, the advantages and disadvantages of each instrument.

Overall, the UK's experience and knowledge in the design and behaviour of dams have been primarily with embankment type dams. This stems from the fact that the majority of the dams within the scope of the reservoir legislation act are embankment type dams (some 2,000 dams). In particular, UK engineers have acquired experience in monitoring embankment dams with very wet clay cores known as puddle clay cores (Charles and Tedd, 1991). A UK survey of embankment dams concluded that the majority of the 2,000 embankment dams (i.e., approximately 1,300 dams) have puddle clay cores (Millmore and Charles, 1988).

The types of data analysis applied to British dams are traditional in the sense that they are very simple and straight forward. In most cases the results are presented in tabular or graphical form. For example, the deflections of embankment dams may be graphed as a function of the reservoir level and those for concrete dams as a function of time.

If the reader is interested in some examples of the types of instruments and survey techniques used to monitoring existing concrete and embankment dams and their associate problems, he or she is referred to the case studies cited by Davie and Tripp (1991), Gosschalk et al. (1991), Johnson et al. (1979) and Ferguson et al. (1979).

3.16 USA

There are as many as 5,469 large dams registered in the USA (Sharma, 1992). According to Sharma (1992), current information regarding how many dams listed in the United States

Committee on Large Dams (USCOLD) Database are instrumented is not available. However, there are a total or 764 dams owned by the Federal Government that are being monitored: 475 by the Corps of Engineers (COE); 265 by the US Bureau of Reclamation (USBR); and 24 by the Tennessee Valley Authority. Whether or not these structures are being properly maintained and monitored is an ongoing concern of organizations such as the USBR, Association of State Dam Safety Officials (ASDSO), Interagency Committee on Dam Safety (ICODS), Federal Emergency Management Agency (FEMA), USCOLD and COE.

Sharma (1992) suggests that the monitoring instrumentation design is entirely system dependant (i.e., the expected behaviour of the dam largely depends on the dam-type and its interaction with foundation and surrounding geological environment). Therefore, there cannot be rigid standards that are applicable to all dams for monitoring their performance; there can be only guidelines and considerations. Some of the guidelines and manuals that are currently used in the US are:

1. USCOLD Publication, *General Considerations Applicable to Performance Monitoring of Dams*, December 1986.
2. *Concrete Dam Instrumentation Manual*, US Bureau of Reclamation (1987), Denver, Colorado.
3. *Embankment Dam Instrumentation Manual*, US Bureau of Reclamation (1987), Denver, Colorado.
4. *Instrumentation for Concrete Structures* (September 1980). Engineer Manual, EM 1110-2-4300, US Corps of Engineers, Washington, D.C.
5. Instrumentation of Earth and Rockfill Dams: Part 1 of 2, *Groundwater and Pore Pressure Observations*, (31 August 1971) and Part 2 of 2, *Earth Movement and Pressure Measuring Devices*, (19 November 1976). Engineer Manual, EM 1110-2-1908, US Army Corps of Engineers, Office, Chief of Engineers, Washington, D.C.
6. *General Considerations on Reservoir Instrumentation*, report by USCOLD Committee on Measurements, 1979/1981.
7. *Seismic Instrumentation in Dams*, USCOLD Committee on Earthquakes, April 1975.
8. Dunnicliff, J. (1988). *Geotechnical Instrumentation for Monitoring Field Performance.*

John Wiley & Sons, New York.

In the USA, the concern for the aging of dams by some federal agencies dates back to the mid 1960s. The concerned agencies established a periodic inspection program to evaluate the conditions of their dams which has contributed to the current active, continuing program of dam safety evaluation in the federal department. However it was not until after a number of tragic dam failures that Congress passed legislation in 1972, known as the *National Dam Inspection Act*. This act called for the inventory and inspection of all non-federal dams. Furthermore, it was not until another tragic event, the Teton Dam failure in 1976, that funds were actually allotted to perform the inspections under the 1972 Act. Immediately following the Teton Dam disaster the Federal Government convened a team of specialists to develop guidelines known as the "Federal Guidelines for Dam Safety." In 1979 the President of the USA implemented these guidelines and FEMA was charged with monitoring conformance to these guidelines. At approximately the same time, after the failure of the Teton Dam, the President directed the COE to up-date the dam inventory and inspect about 9,000 non-federal large and small dams classified as being hazards (Duscha, 1984). To date the COE National Dam Inspection Program has been one of the most significant development in dam safety efforts. Another survey of a review of state dam safety programs conducted in 1982-1984 by the University of Tennessee disclosed that over half (26) of the states still did not have adequate dam safety legislation and adequate resources and personnel to conduct effective and sustainable dam safety programs. An informal survey concluded that these statistics were still viable in 1989 (Ellam, 1990). The newest act that is in effect in the USA is the *Dam Safety Act of 1986* (Title XII of P.L. 99-662) (ASDSO Newsletter, 1989).

At the Fourteenth International Congress on Large Dams in 1982 in Rio de Janeiro, A.W. Wahler of the USA expressed that one of the problems in his country is that earth dams are often delegated to junior engineers because senior developers consider them to be simple. He also added that a number of government agencies in the USA were responsible for designing earth dams without being specialist in the subject (IWP & DC, 1982). With approximately 86% of the 5459 large dams in the USA being embankment type dams one may conclude that W.A. Wahler's

statement is a valid explanation as to why such an exuberant number of American dams (9,000) are considered unsafe by the COE's guidelines.

According to Leps (1985), history of dam failures in the USA has demonstrated that the instrumentation provided in each case was not selected, installed, observed, or evaluated in a sufficiently timely manner, or with the skill judgment required, to permit adequate forecasts of, and offsetting measures against inadequate safety. Leps (1985) supports this statement by citing a number of cases where this is a fact, one being the Teton Dam in Idaho. The only instrumentation at the time of Teton Dam failure consisted of nine surface bench marks on the dam and nineteen deep observation wells in bedrock in the region adjacent to the dam and reservoir. There were no piezometers, no internal settlement or deformation devices and no provision for monitoring leakage except visually (Leps, 1985).

Despite these statistics there has been encouraging developments in dam safety within the past few years. In 1984 a constitution and by-laws were adopted by several states to form ASDSO. With respect to dam safety, ASDSO's mandate is to provide information and assistance to state dam safety programs and to improve the efficiency and effectiveness of the programs. Since its establishment, ASDSO has expanded to include 48 States and two territories. The association has sponsored national meetings on an annual basis where the use of innovative ideas and technology in dam rehabilitation projects is presented and exchanged. In addition, organizations such as FEMA has supported several projects aimed at enhancing the state dam safety programs including the development of a Model State Program for dam safety and the establishment of technical groups to consider special issues (Ellam, 1990). Another important event that is worth noting is the signing of a Memorandum of Understanding (MOU) in 1989 between COE and FEMA to transfer funds authorized in the *Dam Safety Act of 1986* to update the National Inventory of Dams. Under the MOU, FEMA's goal is to eventually establish a National Database consisting of both federal and non-federal owned dams (ASDSO Newsletter, 1989).

As the concern for dam safety in America grew it became apparent that there was little comprehensive dam safety training available. Recognizing this predicament, an ad hoc steering

committee comprising of USBR, COE, FEMA, ASDSO and USCOLD was established in 1985 to explore innovative solutions to the training needs problem. The ideal solution adopted by the steering committee was to design a self-instructional training package. It was argued that self-instructional training is capable of reaching a very large and broad audience and can be readily tailored to the needs of the individual learner. The newly established dam safety training program was given the name Training Aids for Dam Safety (TADS) and obtained the support of fourteen US Federal agencies within ICODS (Veesaert, 1990).

The TADS program consists of three components comprising of a series of modules with workbook and texts, supplemented with videotapes. The first component is entitled "Safety Inspection of Dams" which includes modules on how to prepare for, conduct, and document the inspection of different types of embankment and concrete dams. Included in this component is also a module on the instrumentation of embankment and concrete dams. The second component, "Dam Safety Awareness, Organization and Implementation," consists of a series of modules on the importance of dam safety, dam ownership responsibilities and liabilities, and how to organizing dam safety and operation and maintenance programs, and emergency action planning. The third and last component, "Data Review, Investigation and Analysis, and Remedial Action for Dam Safety," outlines the dam safety process and, how to evaluate hydrologic and hydraulic accuracy, concrete and embankment dam stability, and deformation and seepage conditions. These modules are available through the supporting US Federal agencies or through USBR (Veesaert, 1990).

In the USA, the monitoring of the performance of dams for their structural safety by means of external and internal structural measuring instrumentation dates back to 1930, as exemplified by the Hoover Dam (1936) monitoring system. However, for the reasons stated earlier, it is apparent that this approach was not applied to the majority of the existing dams in the USA. The need for the surveillance of dams and the consideration applicable to monitoring and assessing the structural safety of the dam and its foundation are presented in the USCOLD publication entitled "General Considerations on Reservoir Instrumentation", written by the USCOLD Committee on Measurements 1979-1981 (ICOLD, 1989). The aim of the publication is to provide federal and

non-federal agencies with a monitoring program that will ensure that the dam is safe and operating as expected. A summary of the publication was presented to the ICOLD Committee on Monitoring of Dams and their Foundations and has been published in the ICOLD (1989) as part of the Committee's report. The following discussion will outlined some of the major considerations of the USCOLD's report.

Overall, the USCOLD's report includes the following (ICOLD, 1989):

1. The purpose and need for dam surveillance.
2. For embankment and concrete dams, it recommends the quantities to be measured and the instruments and measuring methods to be used to measure those quantities (see Table 3.23).
3. The procedures to observe when designing a surveillance system (e.g., the need to establish the purpose of the instrumentation, the steps required when selecting instruments, and listing the purpose of each instruments).
4. Recommendations of the monitoring frequencies at the different phases of the project (see Table 3.25 of this report).
5. Considerations concerning data acquisition, processing and presentation of results, including requirements for the personnel responsible for these readings.
6. An example of the principles, process and situations which should be considered in the evaluation of any data set to determine the structural performance of the dam.
7. A guide to some of the factors that should be considered when selecting an automated system for a dam.

The types of instrumentations and monitoring techniques for both embankment and concrete dams recommended by USCOLD are listed in Table 3.23. In Table 3.23, the USCOLD suggests that a surveillance system for all dams should comprise of a combination of geodetic, geotechnical and structural monitoring instrumentation. Moreover, Tables 3.24a and 3.24b illustrate that a number of these instruments have already been installed in some of USA's existing dams. With respect to geodetic methods, particularly when performing precision measurements with Electronic Distance Measuring Equipment (EDME), the USCOLD strongly stresses the need for

a qualified person to make the measurements. Some of the new monitoring methods developed in the USA include the Streaming Potential Method and the Subsurface Temperature Monitoring (Thermotic Monitoring) for monitoring seepage (*ICOLD*, 1989).

Table 3.23
Instrumentation and Monitoring Techniques
Recommended by USCOLD
(after ICOLD, [1989, pp. 287-292])

Quantities Measured	Instruments	E	CG	RCCG	CA	CB	C/E
1. MOVEMENTS							
Horizontal and vertical translation	Precision theodolite, EDM, inclinometer	X	X	X	X	X	
Rotation	Tiltmeters	X	X		X	X	X
Relative	Strain detection devices including joint meters, extensometer and a variety of crack monitoring devices	X	X	X	X	X	X
Strain	Strain meters (e.g., Carlson elastic wire type)		X	X	X	X	
Differential between zones		X					
At joints or at cracks in concrete			X	X	X	X	
2. STRESS							
	Gloetzl flat plate, carlson stressmeter, Goodman flat jack, (strain meters converted into stress)	X	X	X	X	X	
3.GROUND WATER AND WATER PRESSURE							

Quantities Measured	Instruments	E	CG	RCCG	CA	CB	C/E
Pore water pressure	Open stand pipes or wells or by piezometers systems of either the open or closed pipe. Closed system piezometers include hydraulic or pneumatic type.	X					
Uplift pressure		X	X	X	X	X	X
Ground water elevation		X	X	X	X	X	
Seepage Movements	Weirs (90° V-notch, rectangular or trapezoidal type), flowmeter, parshall flumes, calibrated containers, thermatic surveys and self potential measurements.						
* Phreatic surface		X					
* Discharge amounts		X	X	X	X	X	X
Analysis of Collected Seepage							
* For solids		X					
* For chemical compounds		X	X	X	X		
Detection of seepage paths		x	x	x	x	x	x
4. TEMPERATURE							
Of the water (at various levels, in the reservoir and below the dam)	Thermometers	x	x	x	x	x	
Of concrete at various depths at the mass	Carlson type resistance thermometers, face thermometers		x	x	x	x	
Of atmosphere	Thermometers *	x	x	x	x	x	x
Of soil or foundation mass	Carlson type resistance thermometers	x	x	x	x	x	
5.SEISMIC EFFECTS							
Accelerations	Seismographs and strong motion accelerographs	x	x	x	x	x	x
Displacement		x	x	x	x	x	x

E = Earth
RCCG = Roller Compacted Concrete Gravity
CB = Concrete Buttress

CG = Concrete Gravity
CA = Concrete Arch
C/E = Concrete and Earth

Table 3.24a
Dam Monitoring Instrumentation
Applied to Existing Concrete Dams in the USA
(from ICOLD, [1989, p. 316])

Name (year)	Height/Length (m)	INSTRUMENT TYPE							
		Plumbl.	Uplift Press.	Collim.	Found. def.	Embed. Instr. (strain, temp)	Drain Flows	Seepage Meas.	Others
Crystal (1976)	98/194	16	-	3	16	28	54	13	Slide meas. strain gauges
East Canyon (1966)	79/133	-	-	4	10	-	-	-	Triangul.
Flaming Gorge (1964)	153/392	24	13	8	-	1,078	-	15	16 Therm.
Grand Coulee Forebay (1974)	55/357	24	33	11	10	23	37	-	Whittemore pts; deflectometers; therm.
Glen Canyon (1964)	216/476	25	38	-	12	1,800 +	16	-	Triangul; climatological
Hungry Horse (1953)	172/645	12	50	-	-	464	139	-	Climatological
Monticello (1957)	93/312	6	-	-	-	162	-	-	EDM; triangul.

Name (year)	Height/Length (m)	INSTRUMENT TYPE							
		Plumbl.	Uplift Press.	Collim.	Found. def.	Embed. Instr. (strain, temp)	Drain Flows	Seepage Meas.	Others
Morrow Point (1968)	143/221	26	-	5	3	1,118	36	-	Power plant monuments; slide monitoring
Nambe Falls (1976)	46/98	-	10	6	-	60	21	11	Embedded meas: flat jack press. (12)
Pueblo (1975)	58/3109	6	10	6	6	-	8	-	Buttress movement (12), EDM-20 embedded meas.
Yellowtail (1966)	160/451	9	23	3	9	1,650 +	-	193	Obs. wells 45
Clarance Canon (1984)	138/1940	2	42	-	-	47	-	-	Trilat. 43 pts
Dworshak (1973)	218/1000	2	55	52	9	790	-	4	Seismograph (4)
New Bullards Bar (1969)	193/716	4	18	-	39	518	-	15	Triangul. 13 pts; jointmeters 172

Table 3.24b
Dam Monitoring Instrumentation
Applied to Existing Embankment Dams in the USA
(from ICOLD, [1989, p. 317])

Name (year)	Height/ Length (m)	INSTRUMENT TYPE						
		Pnuem. Piezom.	Stand-pipe Piezom.	Inclin.	Vibr.-Wire Piezom.	Meas. Pts	Seepage Meas.	Others
Calamus (1985)	29.3/2195	48	106	3	16	66	25	Baseplate 12; pneumatic s-ettl. sensors 48; thermistor 111
Choke Canyon (1982)	43.1/5631.4	none	53	none	-	96	none	Baseplate 7
McGee Creek (*)	50.0/6000	77	41	7	-	57	none	Pneumatic settlement sensors 8; total press. cells 24
McPhee (1984)	82.3/396.3	96	22	7	-	40	none	Pneumatic settlement sensors 64; extensometers 2; strong motion 5
Navajo (1963)	123/1112	none	48	none	-	60	7	Hydraulic piezometers 40; internal vert. move-ments 2; water analysis 11; horizontal drain 3
Palmetto Bend (1980)	21/13,904	none	60	4	-	3	128	
Red Fleet (1980)	44/518	27	38	9	-	70	2	Horizontal drains 100; tunnel drains 30

Name (year)	Height/ Length (m)	INSTRUMENT TYPE						
		Pnuem. Piezom.	Stand-pipe Piezom.	Inclin.	Vibr.-Wire Piezom.	Meas. Pts	Seepage Meas.	Others
Ridgeway (*)	69.2/740.9	68	53	14	8	86	2	Total pressure cells 13; exte-nsometers 16; strong motion 6
San Justo (*)	43/220.1	none	30	23	32	100	15	
San Luts (1967)	115/5669	none	61	19	70	250	13	Hydraulic piezometers 119; internal vert. move-ments 4; baseplate 3
Sugar Pine (1980)	58/183	21	4	5	-	20	1	Hydraulic piezometers 30; total pres-sure cells 29; extensometers 12; internal vertical move-ments 1
Clarence Canon		62	20	9	21	28	-	Carlson soil pressure cell 1; Carlson electrical piezometers 5

(*) Signifies that it the dam was under construction at the time the table was published.

Although it may seem elementary, a significant point with respect to the instrumentation system design is that the personnel selected to be responsible for the instrumentation should be able to answer the question: "Is the instrument functioning correctly?" In doing so, they should be capable of checking for gross error by a simple visual means or if required by more extensive means, and periodically calibrate and maintain the instrumentation. Furthermore, these persons should be thoroughly experienced, and capable of fully understanding the purpose and importance of the instrumentation. The USCOLD's report continues on to suggests that this group be headed

by a senior Civil Engineer who is intimately familiar with the instrumentation system, the dam and its structural behaviour (ICOLD, 1989). It would appear that the USCOLD is trying to avoid the situation that transpired with the Waco Dam failure in Texas from reoccurring. Waco Dam is an example of a failure where significant movements were noted from a review of the construction survey data, but at the time was ignored and explained away as being a survey error. Later, investigations revealed that the failure was caused by a slippage in the foundation clay shale about 15 meters below the ground (Stroman and Karbs, 1985).

The USCOLD's recommended observation schedule from which a basic framework can be formulated for a specific monitoring schedule compatible with the types of instruments installed in the project are listed at Table 3.25. The observation schedule indicates the monitoring frequencies to be observed over the four basic stages of the life of a dam; during construction, first filling, initial holding (if applicable), during the first year and after the dam attains a stabilized pattern of behaviour. Also included, as a note, are the measurements that should be observed before construction begins (ICOLD, 1989). For comparison an actual observation schedule that is being implemented on Monticello earth dams in South Carolina is provided in Table 3.26.

To enable a better assessment of instrument performance and to increase the confidence in the readings, USCOLD suggests that the monitoring system should have some redundancy. This may be accomplished by installing instruments with different types of sensors close to each other (e.g., in the case of piezometer installations, flushable hydraulic cells can be installed beside vibrating wire instruments). The necessary data for the safety evaluation of the dam should be presented in tabular or graphical form and compared with the predicted behaviour. Measured values of response patterns (deflection, seepage, uplift) plotted against time is considered be one of the key end product of data processing. If dam owners do not have the expertise to perform the data processing and analysis, they are encouraged to seek the guidance of federal agencies such as USBR, COE and other larger agencies. Generally, according to the US National Report (ICOLD, 1989), these agencies own and operate many dams and have well established in-house instrumentation and dam performance evaluation groups staffed with experienced engineering

personnel. There is, however, some doubt whether this statement represents the true situation, particularly in the area of geodetic monitoring.

Automation of data collection in American dams is increasingly being used primarily because of: (1) the decrease in cost of the technology, (2) the increased reliability of the systems, (3) the greater availability of electronic sensors for making measurements, and (4) the increased cost of labour for monitoring. The USCOLD provides a lists of instruments which they consider can be readily automated, and a list of dams owned by USBR and others that have been installed with an automated data acquisition system (ICOLD, 1989). The reader is referred to Walz (1989) for additional information on the automation of data management in the USA. Walz (1989) includes the types of system components, hardware and software requirements that need to be considered when implementing such a system. He also provides a table suggesting the types of instruments that can and cannot be easily automated.

Table 3.25
USCOLD Instrument Monitoring Schedule
(from ICOLD, [1989, p. 325])

PERIOD	TYPE OF MEASUREMENT		
	DEFLECTION & DEFORMATION	STRESS, STRAIN & TEMPERATURE	SEEPAGE, PIEZOMETER LEVELS
During Construction	PL - weekly	SS - weekly	U - weekly
	SL - prior to filling	SM - weekly	D - weekly
	FD - weekly	T - weekly	P - weekly
	NP - weekly		

PERIOD	TYPE OF MEASUREMENT		
	DEFLECTION & DEFORMATION	STRESS, STRAIN & TEMPERATURE	SEEPAGE, PIEZOMETER LEVELS
First Filling	PL - daily during fill or each specified rise SL - once after reservoir reaches level to be maintained FD - daily during fill or each specified rise MP - daily during fill or each specified rise	SS - once each specified rise SM - once after reservoir reaches level to be maintained T - once after reservoir reaches level to be maintained	U - following filling D - following filling unless un-anticipated flow is encountered P - daily during fill or once each specified rise
Initial Holding (if applicable)	PL - daily for first wee, weekly thereafter SL - monthly FD - weekly MP - weekly unless creep is indicated	SS - weekly SM - weekly T - weekly	U - daily for first wee, weekly thereafter D - weekly P - daily for fist wee, weekly thereafter
Subsequent First Year's Operation	PL - bi-monthly SL - quarterly FD - monthly MP - monthly	SS - bi-monthly SM - bi-monthly T - bi-monthly	U - weekly D - weekly P - weekly
After Dam Attains Stabilized Pattern of Behaviour	PL - monthly SL - annually at high reservoir FD - monthly MP - monthly	SS - monthly SM - monthly T - monthly	U - weekly D - weekly P - weekly

Note:

Pre-construction Observation

* Geodetic - once before start of construction
* Groundwater levels - once before start of construction
* Seismic activity - early before construction to establish ref base

PL - Plumblines
SL - Survey Transverse,
SS - Stressmeters
SM - Strainmeters
U - Uplift Pressure
D - Seepage

Triangulation | T - Thermometers | P - Piezometers
FD - Foundation Deformation Meters
MP - Multiple Position Extensometer

Table 3.26
Frequency of Instrument Readings and Dam Surveillance
Fairfield Pumped Storage Facility
South Carolina (USA)
(from Massey, [1982, p. 1155])

Instrument	Before Filling	During Filling	After Filling	Remarks
Accelerometer			Automatically signals powerhouse following earthquake larger than .0045g.	
Water level recorder piezometers			weekly or within one hour following the triggering of the accelerometer	
Permanent piezometers and observation Wells (a)	bi-weekly	twice/week with reservoir below El. 400 daily with reservoir above El. 400	bi-weekly: continue for 2 months after reservoir is first filled weekly thereafter (b)	
Horizontal drains, intake structure, vertical pipe drains, springs, flow at all toe drainage weirs, downstream weirs and drains, and relief wells	bi-weekly	daily	same frequency as tabulated above for permanent piezometers (c)	reservoir perimeter springs & wells are read monthly
Turbidity measurements	bi-weekly	daily	monthly	

Instrument	Before Filling	During Filling	After Filling	Remarks
Settlement plates and borros anchor points	bi-weekly	twice/week	bi-weekly: continue for 2 months after reservoir is first filled monthly thereafter	
Crest monuments, intake monuments, and penstock slope monuments and targets	bi-weekly	weekly	bi-weekly: continue for 2 months after reservoir is first filled monthly: after 2nd full reservoir month	
Slope indicator	bi-weekly	weekly	bi-weekly: continue for 2 months after reservoir is first filled quarterly thereafter	readings were discontinued in 1980 because of damage to pipe
Lake level and rainfall	daily	daily	daily	
Slope targets	bi-weekly	weekly	bi-weekly: continue for 2 months after reservoir is first filled	penstock slope targets continued monthly
Surveillance of reservoir shoreline	N/A	daily	quarterly (d)	
Surveillance of crest, u/s face, d/s face and area d/s of toes	after completion of construction	weekly: with reservoir below El. 400 daily: with reservoir above El. 400	concurrent with readings of the permanent piezometers	
General routine inspection of all Monticello Dams	after completion of construction	daily	daily	

(a) The number of permanent piezometers being monitored during the life of the facility may change.

(b) If an earthquake causes the accelerometer to trigger and the permanent and continuous water level recorders of Dam B indicate a water level change equal to or greater than one foot compared with the previous reading, frequency of the readings will increase to daily or as requested by the engineer.

(c) If an earthquake causes the accelerometer to trigger and the flow or turbidity changes are noticed, frequency of readings or observations will change to daily or as requested by the engineer.

(d) Frequency may be adjusted depending on rate of erosion.

In addition to the information provided by USCOLD's report in ICOLD (1989), there are a number of articles related to dam safety and monitoring that have been published by different experts and members of other organizations in the USA. These include reports on the implementation of monitoring instruments (Leps, 1985; Lytle, 1985), monitoring systems and instrumentation used on existing dams (Moore and Kleber, 1985; Stroman and Karbs, 1985; Massey, 1982) and case studies of some of the problems encountered with embankment and concrete dams (Davis et al., 1991; Fiedler et al., 1991; Kelly, 1991; Thompson et al., 1991; Murray and Browning, 1984; Abraham and Sloan, 1979; Fetzer, 1979).

Other excellent sources of information are the aforementioned USBR guides on the instrumentation of concrete and embankment dams entitled, "*Concrete Dam Instrumentation Manual*" and *Embankment Dam Instrumentation Manual*" (Bartholomew and Haverland, 1987; Bartholomew et al., 1987). The manuals not only provide information on the types of instruments and monitoring techniques currently used by USBR but for each instrument the manuals include: their advantages and limitations, samples of data and analysis from existing dams, a list of sample specifications, and guidelines on the frequency of the measurements. These manuals have been prepared primarily for USBR personnel to provide them with information on the installation, operation, and analysis of instrumentation systems of USBR dams, however designers, engineers, surveillance staff, dam owners, and dam safety personnel within the USA or abroad may also use this information (Bartholomew et al., 1987).

4 SUMMARY OF RESULTS, CONCLUSIONS, AND RECOMMENDATIONS

4.1. GENERAL COMMENTS

The scope of work, as listed in the Introduction, has been fulfilled and even exceeded due to a collection of larger than expected amount of information form various countries. However, one has to be very careful in judging the general status of monitoring surveys from the available national reports and answers to the authors' questionnaire. The reports very often generalize the status on the basis of a few selected examples, perhaps the best instrumented, or best analyzed, dams which can create a too optimistic picture in comparison with the average situation. The United States is no exception.

A general conclusion reached from the study is that there is an obvious lack of cooperation, exchange of information, and work coordination between various international study groups on deformation surveys. The same situation exists at the national levels in the United States and Canada. Generally, the structural, geotechnical, and surveying professionals do not exchange information on their techniques and methods used in deformation monitoring and analysis. In practice, this situation slows down the implementation of new developments.

The above comments lead to the following recommendation concerning the United States:

(1) The results of this report should be followed by a more detailed study on the actual status of monitoring individual dams, methods used in the analyses, and educational background of those placed in charge of the monitoring.

(2) The US Committee on Large Dams should place more emphasis than now on the exchange of information between the national associations and agencies involved in dam deformation studies and international bodies, particularly FIG and ICOLD.

(3) The creation of a truly interdisciplinary national committee (perhaps as a part of the US

Committee on Large Dams) consisting of world renowned specialists in geotechnical, structural, and surveying deformation measurements and analyses, to evaluate the overall situation in dam monitoring and prepare nation wide guidelines and manuals on suggested methods.

4.2 STANDARDS AND SPECIFICATIONS FOR DAM MONITORING

Although there are significant differences between various countries in the quality of monitoring deformation, there is no one country which can serve as an example for others concerning all the three main aspects of dam deformation monitoring, i.e. monitoring techniques, design of monitoring schemes, and analysis and management of the collected observations.

A few countries, mainly eastern European countries, during the time when they were still part of the "communist block", including China, developed some national standards and specifications for dam deformations. Unfortunately, these specifications were developed for practically unrealistic conditions under government dictatorship and ownership of all dams in their countries. Although some of the standards and specifications may be technically acceptable, they would have to be carefully reviewed and extensively modified for practical use. Overall, there are no available standards and specifications in any of the reviewed countries which could be recommended for direct adaptation to dam deformation monitoring in the United States. As far as the choice of monitoring techniques and recommended uses of various instruments are concerned, the ICOLD Bulletins no. 41, 60, and 68, and books and manuals, listed in the text, give sufficient guidelines to be followed when using geotechnical instrumentation. Although there are some reputable books on specialized geodetic instrumentation, there is no up-to-date manual or book which discusses all of the aspects of geodetic monitoring surveys, particularly the design and processing of the geodetic monitoring networks.

The above comments lead to the following recommendations:

(1) Although it seems to be unrealistic and, practically, impossible, to prepare overall detailed standards and specifications for dam monitoring at the national level,certain

processes could and, perhaps, should be standardized, particularly:

(a) calibration of instruments, and

(b) procedures for the integrated geometrical analysis and data management from the moment of the data collection, through the reduction of data and trend analysis (including the identification of the unstable reference points) to the determination of the deformation model.

(2) General guidelines, much broader than the aforementioned USBR manuals on instrumentation, should be elaborated at the national level with respect to: design of the integrated monitoring schemes, particularly optimal choice of instrumentation, selection of the parameters to be monitored, the required accuracy, optimal location of the instruments, and frequency of measurements.

(3) Based on the national guidelines, detailed specifications for monitoring and analysis of deformations for each individual dam should be established.

(4) In support of the recommendations (2) and (3), a team of experts in geodetic surveying should be invited to prepare a manual on all aspects of geodetic monitoring surveys including instrumentation, calibration, data reduction and adjustment, integration with other observables, deformation trend analysis, and integrated geometrical analysis.

4.3 MONITORING TECHNIQUES AND THEIR APPLICATIONS

With the recent technological developments in both geodetic and geotechnical instrumentation, at a cost one may achieve almost any, practically needed, instrumental resolution and precision, full automation, and virtually real-time data processing. Remotely controlled telemetric data acquisition systems, working continuously for several months without recharging the batteries in temperatures down to -40°C are available at a reasonable cost. Thus, the array of different types of instruments available for deformation studies has significantly broaden within the last few years. This creates a new challenge for the designers of the monitoring surveys: what instruments to choose, where to locate them, and how to combine them into one *integrated monitoring scheme* in which the geodetic and geotechnical/structural measurements would optimally complement each other.

As far as the actual accuracy of deformation surveys is concerned, the main limiting factors are not the instrument precision but the environmental influences and negligence by the users, namely:

- influence of atmospheric refraction (all optical and electro-optical measuring systems),
- thermal influences, affecting the mechanical, electronic, and optical components of the instruments (in any type of instrumentation) as well as the stability of survey stations,
- local instability of the observation stations (improper monumentation of survey stations and improper installation of the *in-situ* instrumentation),
- lack of or improper calibration of the instruments, and
- lack of understanding by the users of the sources of errors and of the proper reduction of the collected observations.

Most of the above listed effects could be eliminated or drastically reduced if the monitoring surveys were in the hands of qualified professionals.

The problem of calibration is very often underestimated in practice, not only by the users but also by the manufacturers. For long-term measurements, the instrument repeatability (precision) may be affected by aging of the electronic and mechanical components resulting in a drift of the instrument readout. Of particular concern are geotechnical instruments for which the users, in general, do not have sufficient facilities and adequate knowledge for their calibration. The permanently installed instruments are very often left in-situ for several years without checking the quality of their performance.

The last aspect, the lack of understanding of the sources of errors affecting various types of measurements and the proper data handling is, perhaps, the most dangerous and, unfortunately, the most frequent case in measurements of deformations in North America. The measurements and processing of the monitoring data, particularly geodetic surveys, are usually in hands of technicians who may be experienced in the data collection, but have no educational background in handling and reducing the influence of various sources of errors. In this case, even the most technologically advanced instrumentation system will not supply the expected information.

The worldwide review of the monitoring techniques used in monitoring surveys indicates that, generally, there are no significant differences between different countries in the employed instrumentation. There are, however, differences in the accuracy requirements, the required frequency of observations, and in the details concerning the use of the instruments. The readers are referred to Chapter 3 to the tabulated data from different countries. At the time of writing this report all the tabulated data had not yet been fully summarized. Strong biases towards either geodetic or geotechnical instrumentation have been developed in individual countries. The biases are correlated with the level of educational background in geodetic surveys. For instance, in Switzerland and Germany, where the standard of geodetic surveying education and the number of specialists in surveying engineering are high, the geodetic surveying techniques play the dominant role in monitoring large dams. Whereas, in the countries like Italy, France, and U.K., where the education in surveying engineering does not have a long standing tradition, the geotechnical techniques are mostly used. ICOLD seems to be biased towards use of geotechnical/structural instrumentation rather than geodetic. This is dictated, perhaps, by the fact that the members of the ICOLD Committee on Monitoring Dams and their Foundations are predominantly specialists in geoetechnical instrumentation with some obsolete views on the use geodetic techniques. With the exception of few individual dams, there is no country which takes full advantage of the optimal combination of both techniques, i.e. the concept of the *integrated monitoring surveys* developed within the activity of FIG *(Fédération Internationale des Géometres)*. The biases towards one or another type of techniques are obviously produced by a lack of specialists with full knowledge of both geotechnical and geodetic measurements.

The above comments lead to the following recommendations:

(1) The concept of the integrated monitoring systems, in which the geotechnical and geodetic surveying techniques complement each other should be made known to all the owners of large dams through the aforementioned efforts of the national committees on large dams and publication of guidelines.

(2) Monitoring schemes for all new dams should be designed at the design stage of the dam.

(3) As far as the new monitoring technologies are concerned, more research is still needed in:

(a) optimal use of GPS,

(b) automation of data acquisition including the optimal selection of electronic and optical transducers, comparison of their performance (sources of errors, durability), development of calibration methods, and

(c) application of new technologies, for example the use of the optic-fibre sensors, CCD sensors, etc.

4.4 ANALYSIS AND MODELLING OF DEFORMATIONS AND THEIR APPLICATIONS

Over the past 10 years there has been a significant progress in the development of new methods for the geometrical and physical analyses of deformation surveys. FIG has been leading in the developments, particularly in the areas of integrated geometrical analysis of structural deformations (Appendices 4 and 5) and combined integrated analysis (Appendix 7). However, due to the aforementioned lack of the interdisciplinary cooperation and insufficient exchange of information, FIG developments have not yet been widely adapted in practice. Therefore, the general worldwide use of the geometrical analysis methods is still poor, including even the basic analysis of geodetic monitoring networks. The latter is of a particular concern in the United States where there is a shortage of qualified surveying engineers. The authors could cite several examples of large dams in the United states which are monitored by geodetic methods by agencies which do not even have any person on their staff who understands such basic procedures as the least squares adjustment. The worldwide situation with the physical analysis is much better with most countries who utilize both deterministic and statistical methods for modelling and interpreting deformations at various levels of sophistication. However, most countries do not take full advantage of the developments in the integration of the observed deformations with deterministic models to enhance the latter ones. Also only few countries utilize the observed deformations to develop prediction models through the regression analysis. Italy seems to lead in the use of the combined statistical and deterministic modelling. Canada, within the activities of FIG, leads in the development of new concepts in the global integrated analysis (Appendix 6).

For example, the University of New Brunswick has developed new methodology and software DEFNAN for the integrated geometrical analysis and software FEMMA for the finite element deterministic modelling and prediction of deformations.

The above comments lead to the following recommendations:

(1) The analysis of deformation surveys should be in hands of interdisciplinary teams consisting of geoetechnical, structural, and surveying engineers specialized in both geometrical and physical analyses.

(2) More use should be made of the concepts and developed methodologies for the geometrical integrated analysis and combined deterministic/statistical modelling of deformations.

4.5 EDUCATION AND TRAINING

Generally, with the exception of some larger owners (state agencies, large power commissions, etc.) with centralized facilities for the supervision of the monitoring work and analyses, the overall qualifications and educational background of the personnel placed in charge of monitoring surveys, seem to be inadequate, particularly in the areas of data processing and analyses. This is evident not only in the third world countries but also in some advanced industrialized countries including the United States and Canada. For example, there are only two universities in North America, both in Canada, the University of New Brunswick and the University of Calgary, which offer a broad specialization in engineering surveys of high precision and teach students how to optimally use both geodetic and geotechnical monitoring techniques. Unfortunately, the small supply of graduates from their surveying engineering programs is well below the actual needs. On the other hand, students in civil engineering programs, have very little exposure to the newest developments in instrumentation, calibration, and analysis of measurements.

The above comments lead to the following recommendations regarding the situation in North America:

(1) The educational background of those involved in the design and analysis of deformation

surveys must be reviewed and steps must be undertaken to improve the situation by organizing intensive courses and short term training programs.

(2) Larger owners of large dams should delegate individuals with engineering degrees to specialize at a master of engineering level in engineering surveys and in the analysis of deformation measurements. The retrained persons should be placed in supervisory and quality assurance positions regarding monitoring surveys at the dams belonging to their agency.

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